MINISTRY OF HOUSING AND LOCAL GOVERNMENT

Technical Committee on Storm Overflows and the Disposal of Storm Sewage

Final Report

LONDON
HER MAJESTY'S STATIONERY OFFICE

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1970

The Technical Committee on Storm Overflows and the Disposal of Storm Water was appointed by the Minister of Housing and Local Government on 20th May 1955 with the following terms of reference:

"To study and report upon practice relating to storm overflows on sewers and the disposal of storm water and to make recommendations."

The term "storm water" was subsequently amended to "storm sewage" which was considered to be more descriptive of the mixture of sewage and surface water carried by these sewers during storms. It was also agreed that the terms of reference should include a study of the provision and operation of storm tanks for the partial treatment of storm sewage at the treatment works.

Membership of the Committee from its inception has been as follows:

H. W. Coales, Esq., C.B.E., M.C., F.I.C.E. (Chairman to December 1956)
G. S. Wells, Esq., C.B.E., M.C., M.I.C.E. (Chairman from January 1957, died May 1962)
R. A. Elliott, Esq., C.B.E., B.Sc., F.I.C.E. (Chairman from May 1962)
P. Ackers, Esq., M.Sc. (Eng.), M.I.C.E. (from April 1960)
J. S. Alabaster, Esq., B.Sc. (from December 1964)
F. H. Allen, Esq., M.A., M.A.I., F.I.C.E. (to March 1960)
F. W. Allen, Esq., F.R.I.C., F.I.W.P.C., A.M.C.T.
J. B. Bennett, Esq., C.B.E., F.I.C.E., M.I.Mun.E.
C. D. C. Braine, Esq., B.Sc., M.I.C.E. (died January 1961)
J. T. Calvert, Esq., C.B.E., M.A., F.I.C.E., F.R.I.C.
J. B. Dempster, Esq., B.Sc., F.I.C.E., F.I.W.P.C.
Dr. A. L. Downing, M.A., D.Sc., A.M.I.Chem.E., F.I.W.P.C., F.I.Biol., F.I.P.H.E. (from May 1965)
H. Foster, Esq., M.I.C.E., M.I.Mun.E. (to December 1958)
A. L. H. Gameson, Esq., M.A. (from September 1963)
A. N. Gardiner, Esq., M.I.C.E. (to July 1956)
Dr. A. Key, C.B.E., D.Sc., Ph.D., F.I.W.P.C.
W. F. Lester, Esq., B.Sc., F.R.I.C., F.I.W.P.C.
M. Lovett, Esq., O.B.E., B.Sc., F.R.I.C., F.I.W.P.C.
W. H. E. Makepeace, Esq., F.I.W.P.C., F.I.P.H.E.
J. B. Murray, Esq. (to November 1960)
H. R. Oakley, Esq., M.Sc.(Eng.), F.I.C.E., A.M.I.W.E., M.A.S.C.E. (from June 1961)
F. T. K. Pentelow, Esq., M.A. (to December 1964)
A. E. J. Pettet, Esq., M.A., F.I.W.P.C. (to May 1965)
H. A. Sneezum, Esq., M.I.C.E., M.I.Mun.E.

Secretaries:

C. J. Pearce (to October 1955)
H. R. Pollitzer (from July 1956 to October 1958, and from June 1960 to March 1962)
H. A. M. Cruickshank (from December 1958 to April 1960)
G. H. Chipperfield (from May 1962 to August 1962)
P. McQuail (from August 1962 to October 1964)
F. G. Rickard (from November 1964 to April 1968)
Mrs. J. Ash (from May 1968)

Technical Secretaries:

J. W. M. Hawksworth, F.I.C.E., F.G.S. (to April 1960)
M. W. Summers, F.I.C.E., M.I.W.E., M.I.W.P.C. (from June 1960 to December 1966)
A. J. Herlihy, B.Sc., M.I.C.E. (from January 1967)

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FINAL REPORT OF THE TECHNICAL COMMITTEE ON STORM OVERFLOWS AND THE DISPOSAL OF STORM SEWAGE

To the Right Honourable Anthony Greenwood, M.P.,
Minister of Housing and Local Government

Sir,

We have the honour to submit to you our final report. We have met as a full Committee on forty-two occasions and there have been other meetings by Sub-Committees. We have also carried out an extensive programme of experimental and special studies and these are described in the report.

This report supersedes the Interim Report which we submitted in May 1963, when much of the work associated with our experimental studies still remained to be done.

CHAPTER 1. INTRODUCTION

Historical note

1. Although sewerage systems are known to have existed in many parts of the world in ancient times, it is only necessary to look back a hundred years or so to find sanitary conditions in this country which were completely primitive.

2. In the early part of the 19th century, such sewers as existed were constructed for surface water only and it was illegal to discharge what is now known as sewage to them. Sewage disposal was achieved simply by moving refuse from the immediate vicinity of the dwelling and any nearby watercourse was regarded as a logical and proper place to dump it. Later, following the introduction of the water closet, domestic sewage was discharged to cesspools and these, in turn, were often connected to the surface-water sewers which conveyed the cesspool liquid to the watercourse.

3. During the Industrial Revolution, with the uncontrolled discharge of industrial wastes as well as sewage, the condition of many rivers deteriorated rapidly, and literature on the subject of pollution control contains graphic descriptions of the conditions that prevailed.

4. Commissions were appointed in 1857, 1865 and 1868 to inquire into various aspects of sewage disposal in towns and the prevention of pollution of rivers; the conclusion reached was that sewage should be collected from individual properties and conveyed to a central point for disposal on land. The sewage farm, which provided settlement and land irrigation, then became the most common mode of disposal.

5. The publication of the reports of the various Commissions was followed by the passing of the Public Health Act 1875 and the Rivers Pollution Prevention Act 1876; these consolidated the law relating to sewage disposal and river pollution. Under the Act of 1876 it was deemed an offence to discharge sewage to a river.

6. Meanwhile, developments were taking place in the fields of sewerage and sewage treatment. Foul-sewerage systems were growing out of what, in many cases, were the old surface-water systems which had terminated at their various points of discharge to the rivers, and whilst the sewage farm was the standard method of treatment, the large areas of land needed were becoming more difficult to acquire. This led to a search for forms of treatment requiring less land.

7. Other Commissions and Committees examined special aspects of the problem, but perhaps the next major step was the appointment in 1898 of the Royal Commission on Sewage Disposal which, between 1901 and 1915, published a series of reports on the treatment and disposal of sewage. More will be said later about the content of these reports, but at this point it is sufficient to say that they included recommendations for restricting the quantity of sewage to be treated at sewage works when the flow was swollen by surface-water run-off.

8. A situation was therefore developing where foul-sewerage systems, many of which had grown from somewhat haphazard surface-water drainage arrangements, were required to take greater and greater flows of both foul sewage and surface water as populations and the paving of road surfaces increased, and it is evident that there was a good deal of improvisation to meet changing circumstances. Even today, many towns have a complexity of cross-connections within their sewerage systems that can only have resulted from piecemeal attempts to relieve overloaded sewers by diverting some of the flow into others which at the time had some spare capacity. It was not thought necessary, however, to convey all the flow to the final point of disposal; the cost of building the necessary sewers would often have been prohibitive. Excess flows therefore had to be discharged on the way, in such a manner as to avoid flooding, and the many outlets that had existed on the old systems provided ready-made facilities for the discharge of the excess flows to the nearest watercourse; these outlets were, in effect, the first storm overflows. In the absence of such outlets it was often easy and inexpensive to lay a short length of overflow pipe between a convenient manhole and the nearest watercourse to give the necessary relief.

9. It is important to realise that many of the country's largest systems have developed from such beginnings, and although improvements have been made from time to time by the laying of relief sewers and, in some cases, by excluding surface water from the foul sewers, many authorities have not yet succeeded in overcoming the drawbacks stemming from these origins.

Development of present-day practice

10. Although the dual-purpose sewer has developed from a surface-water carrier (often a culverted stream) into which foul sewage has over the years been progressively admitted, it is customary to regard them today as sewers carrying foul sewage into which surface water is admitted in wet weather. In times of heavy rainfall the sewer is required to convey a mixture of sewage and surface water many times greater in quantity than that of the foul sewage.

11. From the earliest days, it has been generally accepted that it would be uneconomical to design entire sewerage systems to carry such flows to the final point of disposal and it is the usual practice to relieve the system of some of the excess flow at selected points by providing storm overflows. In the majority of cases, these take the form of a device (such as a weir) for separating the excess flow which is then discharged through a pipe from the overflow chamber to the nearest suitable watercourse. By this means it is possible to restrict the quantity finally passed to treatment to something very much less than the maximum reaching the sewers in times of rain, and consequently to limit the size of the downstream sewers to reasonable dimensions.

12. In the later part of the 19th century, the Local Government Board was the central authority responsible for sewerage and sewage disposal and it is understood that one of the Board's "requirements" was that storm overflows should not come into operation until a flow equal to six times the dry-weather flow was being conveyed to the sewage works for treatment.

13. The subject of storm overflows was considered by the Royal Commission on Sewage Disposal appointed in 1898, and among the recommendations contained in the Fifth Report (p. 209), published in 1908, were the following:

It is probably impracticable to dispense altogether with storm overflows on branch sewers, but in our opinion these should be used sparingly, and should usually be set so as not to come into operation until the flow in the branch sewer is several times the maximum normal dry weather flow in the sewer.

No general rule can be laid down as to the increase in flow which should occur in the branch sewers before sewage is allowed to pass away by the overflow untreated. The Rivers Board, or in districts where there is no Rivers Board, the County Council, should have power to require the local authority to alter any storm overflows which, in their opinion, permit of an excessive amount of unpurified sewage to flow over them.

The general principle should be to prevent such an amount of unpurified sewage from passing over the overflow as would cause nuisance.

14. Another "requirement" of the Local Government Board was that flows up to three times the normal dry-weather flow should be given full treatment and flows between three and six times the dry-weather flow should be given land treatment or should be passed through storm filters. On this subject, the Royal Commission's Fifth Report included (p. 210) the following:

We therefore recommend, as a general rule

(*1*) *that special standby tanks* (*two or more*) *should be provided at the works and kept empty for the purpose of receiving the excess of storm water which cannot properly be passed through the ordinary tanks. As regards the amount which may be properly passed through the ordinary tanks, our experience shows that, in storm times, the rate of flow through these tanks may usually be increased up to about three times the normal dry weather rate without serious disadvantage;*

(*2*) *that any overflow at the works should only be made from these special tanks and that this overflow should be arranged so that it will not come into operation until the tanks are full.*

Later in the same Report (p. 233) it was suggested that

In most cases it will probably suffice to provide stand-by tanks capable of holding one quarter of the daily dry weather flow and it will not be necessary to provide for filtering more than three times the normal dry weather flow.

Under the arrangements which we recommend no storm sewage arriving at the outfall works would be discharged without some settlement.

15. Although there was no official publication as such, it is generally believed that the Local Government Board's "requirements" were modified in accordance with the Royal Commission's recommendations, but the Board appear to have adhered to their original "requirement" that, in the absence of any special circumstances, overflow weirs should be fixed so as not to come into operation until the flow exceeded six times the average dry-weather flow (6 DWF). When, in 1919, the Ministry of Health took over the functions of the Local Government Board in relation to sewerage and sewage disposal, these "requirements", although (so far as we know) still not laid down in any official document, were included in what became generally known as the "Ministry of Health Requirements" and, regardless of their obscure origin, they are the standards to which designers have worked for many years. Thus evolved the practice of setting storm overflows at 6 DWF, providing full treatment for flows up to 3 DWF, and providing settlement for flows between 3 DWF and 6 DWF in storm tanks having a capacity of 6 hours' dry-weather flow.

16. Conditions have changed considerably since these "requirements" first became the basis of design practice some 60 years ago. Extension of public water supplies and the more widespread use of water-borne sanitation have resulted in great increases in water consumption and hence in the dry-weather flow of sewage; there has also been a large increase in the discharge of industrial effluents to public sewers. Concurrently, abstraction of water from some rivers has materially reduced their average flow. There has been a big increase in paved areas, and the character of surface water itself has changed, because the run-off from roads used by motor vehicles is very different from that draining from roads used by horse-drawn traffic. Despite these changes over the years, however, the old standards are still generally followed. They have been the subject of criticism but no alternative basis of design has been laid down.

17. Under the Rivers (Prevention of Pollution) Act 1951, it became necessary to obtain the consent of the appropriate river board before making any new, or altering any existing discharge of industrial or sewage effluent to a stream and, in granting consent, the boards were empowered to impose conditions. In issuing consent to the discharge from a storm overflow, it became the practice to stipulate a "setting", usually by requiring no discharge to take place until a specified flow was being passed to the sewage works for treatment.

18. A setting equivalent to 6 DWF was (and still is) frequently stipulated—in line with the old "requirements"—but sometimes there have been differences of opinion as to what should constitute a special case and what should then be the appropriate setting. There was a growing practice among some river boards to ask for overflow settings of 8 DWF or even 10 DWF but there was no uniformity of practice, or knowledge

of the magnitude of the benefits that these higher settings would bring about. There have also been differences in interpreting what is meant by 6 DWF. Some considered DWF in this context to be the average total dry-weather flow, including industrial effluents and infiltration; others designed overflows to pass to treatment six times the average domestic sewage flow, plus a multiple (often 1.1) of an estimated rate of infiltration, plus varying multiples of the average discharge of industrial effluents.

19. It became evident that the basic practice of setting overflows at 6 DWF and indeed all aspects of storm-sewage disposal needed examination, and we were accordingly appointed in 1955 to investigate the problem.

Work of the Committee

20. We have met on forty-two occasions and have had the benefit of over 150 memoranda, which were circulated as Committee documents. We have studied and discussed a wide range of subjects associated with the problem and have obtained from many local authorities and other bodies information on a variety of special aspects such as costs, infiltration, performance of existing installations and effects of pollution by storm sewage and surface water. Many visits were made on our behalf to local authorities and river boards (now river authorities) to inspect installations and to collect information.

21. Early in our work it became apparent that in almost all aspects of the problem there was inadequate information on which to base recommendations and we therefore initiated investigations and other special studies which have much prolonged the work of the Committee. These included:

(1) A survey of storm overflows in 52 local authority areas to obtain information about the numbers and types of overflows in existence and the local circumstances in which they operated, together with the river boards' assessment of their performance;

(2) An investigation (by the Water Pollution Research Laboratory) of the rate of flow and composition of storm sewage in three drainage areas;

(3) Experiments (by the Hydraulics Research Station) on small-scale models of different types of overflow to compare their performance and assess the value of storage under the time-varying flow of storm conditions;

(4) Field-scale experiments (by the Hydraulics Research Station and the Water Pollution Research Laboratory) on different types of overflow, to test their hydraulic efficiency and also their efficiency in limiting the amount of pollution discharged by the overflow;

(5) An investigation of the performance of storm tanks.

22. In 1963 we appointed a Sub-Committee to arrange for and analyse the results of the survey of storm overflows. Membership of this Sub-Committee was: Dr. Key (Chairman), Messrs. Ackers, Gameson, Lester and Oakley. When this work was completed, the same Sub-Committee continued in existence as the Drafting Committee. The Secretary of the Sub-Committee was the Technical Secretary for the time being of the main Committee.

23. We consider it appropriate at this point to explain the background against which our studies have been conducted, to enable the position in which we now find ourselves to be more easily understood.

24. We have already referred to the Royal Commission on Sewage Disposal and we have quoted from their Fifth Report some of the references to storm sewage. Their recommendations so far as storm overflows are concerned were (as we have indicated) of a general nature only, but they were more specific in their recommendations in regard to standards for sewage treatment works effluent as set out in their Eighth Report, published in 1912. It is of interest to recall, however, that what became generally known as "the Royal Commission Standard" for sewage effluent was calculated mainly on the basis of dilution of the effluent with eight volumes of clean river water. Furthermore, the Royal Commission said that, in fixing any special standard, the dilution afforded by the stream is the chief factor to be considered. For a period of about forty years, between the Royal Commission's recommendations and the formation of the river boards in 1950–51, little account was taken of the specified minimum dilution required for a normal effluent though, more recently, river authorities have pressed this consideration when fixing effluent standards. So far as storm-sewage discharges are concerned, the degree of dilution available has not entered into calculations though, if this is a pertinent consideration for treated effluents, it would seem equally relevant to the setting of storm overflows.

25. A need has developed recently for more restrictive standards than those recommended by the Royal Commission. Attention has turned to factors in addition to dilution which influence the ability of a river to absorb pollution without adverse effects. However, it was not until 1966 that an official publication[1] was issued which encouraged departure from the traditional dilution approach. This trend is likely to continue now that the detailed study of the behaviour of rivers and their ability to deal with pollution has become practicable with the development of equipment capable of producing continuous records of certain criteria of river-water quality.

26. Thus, in 1955 when we began our studies, standards for treated effluents took little account of any factor besides dilution, and standards for storm-overflow settings rarely took account even of that.

27. We believed it probable that the reaction of a river to intermittent pollution would be different from its reaction to continuous discharges; we were also

aware that there were other important considerations, such as the character of the river in regard to its ability to maintain a sufficient content of dissolved oxygen to meet the needs of polluting discharges, and the proximity of storm overflows to water intakes. We should have liked, therefore, to have initiated a series of field investigations to study this aspect in parallel with those referred to in item (2) above, but we found this to be impossible for practical and economic reasons.

28. As an alternative, we conceived the idea of arranging for the controlled discharge of sewage of measured strength and volume into a river, with arrangements for studying the river regime before, during and after discharge. The possibility of an investigation on these lines was still under active consideration when, in 1963, we submitted an Interim Report[2] which set out the progress made by the Committee up to that time and the tentative conclusions we had reached from the work already done. However, it proved extraordinarily difficult to find a length of river suitable for the exercise, suitably situated and free from several possible kinds of complication. Also, it seemed that the legal implications of the project might be embarrassing, and we eventually decided, with great reluctance, that we must abandon the idea.

29. At this stage, we concluded that the prospect of filling this gap effectively in reasonable time was remote, and we decided that on completion of studies then in hand, we should submit our final report giving such guidance as we could from the evidence available to us.

References

1. Ministry of Housing and Local Government. Technical Problems of River Authorities and Sewage Disposal Authorities in Laying down and Complying with Limits of Quality for Effluents more Restrictive than those of The Royal Commission (H.M.S.O. 1966).

2. Ministry of Housing and Local Government. Technical Committee on Storm Overflows and the Disposal of Storm Sewage. Interim Report. (H.M.S.O. 1963).

CHAPTER 2. EXTENT OF THE PROBLEM

Sewerage systems at present in service

30. The three types of sewerage system are well known. In the "combined" system, the foul sewage and all the surface water are carried by the same sewer, whereas in the "partially-separate" system, only a proportion of the surface water (usually that from back roofs and yards which cannot conveniently be separated because of the layout of house drains) is discharged to the foul sewer, the remainder being collected in surface-water sewers. In the "separate" system, foul sewage and surface water are, in theory, kept strictly in separate sewers. It is on the combined and partially-separate systems that storm overflows occur.

31. We have described in Chapter 1 how, with rapid expansion of towns, many of the long-established sewerage systems in the country grew out of what had originally been systems of road drains or somewhat haphazard networks of foul and surface-water drains discharging to the nearest watercourse; this situation has led to the widespread occurrence of the combined and partially-separate systems. An indication of the scale on which they are to be found is shown in Table 1 from which it will be seen that the majority of authorities covered by the survey have wholly dual-purpose sewerage systems and only 8 per cent have completely separate foul-sewerage systems.

32. The information in Table 1, assuming the survey to be representative, gives some guidance on the number of people in this country at present served by systems of combined or partially-separate sewers and we estimate that, in England and Wales, approximately 15 million people are served by combined systems and a further 21 million by partially-separate or mixed systems.

33. Although the separate system has for some years now been widely used, it is not of such recent origin as is often supposed; it was in fact known and in use in the early part of this century. The Royal Commission discussed its merits in their Fifth Report (1908). They did not make any recommendations for or against its use but brought out a number of points which, in their view, could weigh against its adoption. The main points made were the increased cost of two sets of sewers and drains, the polluting nature of the surface water itself, and the danger of accidentally connecting a foul drain to a surface-water sewer. These points are in the minds of designers at the present time, and whilst there are many who are firmly of the opinion that the separate system should be universally adopted and who advocate a policy of progressive separation of existing dual-purpose systems, there are others who believe that a partially-separate or a combined system with carefully sited and correctly designed storm overflows is often to be preferred.

34. In recent years there has been a strong tendency to adopt the separate system for developments such as new towns and new estates. It is also widely used where sewers are provided for the first time in rural areas. We are unable to say that the benefits derived from separate-system sewerage would always justify the expense of installing the two sets of sewers and drains it requires, but we certainly do not disapprove of this trend. We must point out, however, that it would be wrong to insist that there should be no connections of surface water to foul sewers whatsoever. Situations where the connection of a limited amount of surface water to foul sewers could be justified are discussed in the next section, the limitation being, in our view, that such connections would not necessitate a storm overflow.

Pollution from surface-water sewers

35. It is well known that the water which is discharged from a surface-water outfall sewer is seldom clean. Pollution can arise from a number of causes such as oil, road grit and chemical and other washings from paved areas of industrial premises, and in order to obtain some indication of the incidence of pollution by surface-water sewers, we sent enquiries to all the river boards in England and Wales. The replies showed that many of the boards had encountered such problems, although this should not be taken as implying that the boards preferred the combined system.

TABLE 1. Types of sewerage system found in a sample survey of 226 local authorities in England and Wales (carried out by the Institute of Sewage Purification and published in the Directory of Sewage Works 1963)

Type of sewerage system	County Boroughs	Municipal Boroughs	Urban Districts	Total	Percentage of total
Entirely "separate"	2	7	9	18	8
Entirely "combined"	19	23	24	66	29
Entirely "partially-separate"	8	28	57	93	41
Combination of two systems	4	13	9	26	12
Combination of all three systems	13	6	4	23	10
Total	46	77	103	226	100

36. Apart from the risk of accidental spillage and wrong connections to surface-water sewers, concern was expressed about places such as industrial developments and open cattle markets, from which polluted water can enter surface-water sewers directly from paved areas through surface-water gullies or by seepage through the ground to open-jointed drains.

37. It was evident that there are some areas from which it is desirable that polluted surface water should be discharged to foul sewers and much, if not all of it, retained for treatment, with the sewerage system designed to allow this to be done. On the other hand, surface water gathered from residential, shopping and commercial development is unlikely to be a source of significant pollution, subject of course to the maintenance of adequate safeguards against wrong connections.

38. With these factors in mind, we are of the opinion that the exclusion from foul sewers of all surface water would not be the right policy to recommend. The ideal (disregarding cost) might be a system in which the connection of surface water to foul sewers was only permitted where the surface water run-off was likely to be of a heavily polluting character.

Cost and practicability of improvement

39. We do not know of any attempt in this country to estimate the overall cost of separating surface water from existing combined or partially-separate systems and we do not believe that a reliable estimate could be made without a disproportionate effort to estimate the cost of a statistically significant number of examples. Such figures as we have seen suggest, however, that the order of magnitude would be over £100 per head of population and this is to some extent confirmed by a survey[1] carried out by the U.S. Department of Health, Education and Welfare, covering 15 cities with a total population of 21 million, where the estimated cost of conversion was found to vary from $187 (£78) to $915 (£380) per head with an average cost of $468 (£195) per head. The estimates suggest that to implement a policy of complete separation would involve an expenditure exceeding £3,000 million, and even if this were spread over a period of some thirty years, the cost would probably be in excess of the present annual level of capital investment in sewerage and sewage treatment.

40. Not only would the cost be very large, but the physical problems associated with the introduction of an additional system of sewers into closely developed areas, together with the work of reconstructing private drains, would be enormous and so we conclude that a policy of separation of existing systems would be very difficult to implement and would not be economically feasible. We think that the best that can be achieved is to take such opportunities as occur to divert surface water from existing sewers where alternative methods of disposal are available at reasonable cost and to re-sewer re-development areas on the separate system as opportunity occurs.

41. Accepting that combined or partially-separate sewers (and, therefore, storm overflows) must be retained for many years to come, we next attempted to assess the additional cost that would be involved if a policy were followed of having overflow settings higher than the current practice of 6 DWF. A number of our members examined schemes with which they were concerned, redesigned them for notionally higher settings and estimated the extra cost. Other schemes lodged with the Department were examined in the same way. As would be expected, the results varied widely; the extra cost of sewerage for increasing the setting from 6 to 8 DWF ranged from zero to 30 per cent, and for increasing from 6 to 12 DWF ranged from 12 per cent to 70 per cent. If it were considered necessary to provide storm tanks at the treatment works of sufficient capacity to give the normal period of retention to the higher flows, additional cost would be incurred in providing the greater capacity and it was estimated that, in some instances, this was in excess of the increase in cost of sewerage.

42. So many factors affect the consequent increases in cost, such as sizes of the existing and new sewers, location, depth, ground conditions and amount of pumping, that we do not think that any useful figure can be estimated as an average cost for increasing the amount of storm sewage conveyed to treatment.

The present incidence and performance of overflows

43. When the Committee was appointed, there was virtually no information available on the number of overflows in the country, the proportion considered unsatisfactory or the reasons for dissatisfaction. In an attempt to fill this gap, we invited the river boards to tell us about their experiences with storm overflows and their views on how the storm-overflow problem could be solved. As expected, the replies varied in detail, but the consensus of opinion was that pollution from storm overflows resulted mainly from settings lower than the traditional 6 DWF and that many of the troubles could be solved by setting overflows at 6 DWF or higher, coupled with improved design to minimize discharge of strong storm sewage and gross solids.

44. The boards were asked to give their views on settings higher than 6 DWF, with particular reference to a setting of 8 DWF. Individual opinions varied, but it could be inferred from their replies that such a setting would probably be adequate for the average case. Mention was made, however, of the need for special consideration in individual cases, particularly when there is little dilution available in the receiving stream.

45. More recently we were able to take advantage of the new situation resulting from the passing of the Rivers (Prevention of Pollution) Act 1961 which required, amongst other things, the return of information on all overflows not covered by the 1951 Act, and which provided the river boards with an opportunity to build up information about overflows in their areas.

Although the new Act required only a limited amount of detailed information to be supplied, we thought it worthwhile to initiate a survey.

46. Questionnaires seeking information on a number of aspects of storm overflows were sent to six river boards*—the Great Ouse, Kent, Mersey, Severn, Trent and Yorkshire Ouse. We asked for details of all the known storm overflows in 52 local authority areas selected by us as being representative of the country as a whole. The problem was thought to be more acute in the larger urban areas and, accordingly, the authorities selected included a relatively large proportion of county boroughs and a relatively small proportion of rural districts. As a result, the proportion of overflows in the survey area discharging to polluted rather than unpolluted rivers was probably higher than that for the country as a whole. The 52 local authorities represented 3.6 per cent of all local authorities, and it is estimated that the populations served by main drainage in these areas amounted to 8.2 per cent of the total population on main drainage in England and Wales (excluding the former Administrative County of London which was considered to be a special case).

47. We appreciate that there are a number of potential sources of error in such a survey. For example, the local authorities were not expected to make any special checks on the present dry-weather flows and settings applicable to their overflows, especially in view of the difficulties inherent in the measurement of flow in sewers. The information was therefore based on the authorities' records which may not always have taken account of the most recent changes in conditions. We have therefore to accept that there may be doubts about the accuracy of an appreciable proportion of the information supplied. Furthermore, as there is no generally accepted quantitative criterion by which a given overflow can be classified as satisfactory or unsatisfactory, the answers given in the survey were necessarily the opinions of the officers of each river board, based upon the particular circumstances of each case and after considering the size, location and subsequent use of the receiving watercourse. We could not, however, see any other way to approach such a survey and we have accepted the information given and the opinions offered by the boards as being sufficient for the purpose of the investigation.

48. The survey was conceived as a pilot study which might perhaps be extended. However, after detailed examination of the information provided, we decided that this was sufficient to enable us to gauge the broad extent of the problem and to draw general conclusions, and that the cost of extending the survey to give country-wide coverage would not be justified.

49. The information obtained related to a total of 849 overflows, the number in individual local authority areas varying widely up to a maximum of 140. Settings were reported to vary from as low as 1.5 DWF to over 100 DWF, the dry-weather flow in the sewers at the overflows ranged from under 0.1 to over 10 mil gal per day (m.g.d.)† and the summer flow in the receiving streams varied over a similar range. More than half of the overflows were on sewers carrying a dry-weather flow of less than 0.3 m.g.d.

50. Out of the total of 849 overflows, 317 were considered unsatisfactory. Extending these figures to the country as a whole, it is estimated that there are, in England and Wales, between 10 000 and 12 000 overflows of which about 63 per cent would be assessed by the river authorities as satisfactory and 37 per cent as unsatisfactory.

51. In many cases, more than one reason was given for classifying a single overflow as unsatisfactory, and the information provided is summarized in Table 2.

TABLE 2. Summary of reasons for classifying overflows as unsatisfactory.

Reason	No. of overflows unsatisfactory for each reason (Percentage of total in brackets)	Percentage of 317 overflows unsatisfactory for each reason
A. Stranding of solids in vicinity of watercourse	74 (16)	24
B. Effect on fish, biology	55 (12)	17
C. Operation in dry weather	33 (7)	10
D. Too frequent operation in wet weather	131 (28)	41
E. Combined influence with neighbouring overflows	149 (33)	47
F. Deposits of sludge in watercourse	8 (2)	3
G. Miscellaneous	11 (2)	3

52. The largest single category comprised those unsatisfactory for "combined influence with neighbouring overflows" (Category E). It appeared that, in most cases where a group of neighbouring overflows had an unsatisfactory effect, the river board classified each individual overflow in the group as unsatisfactory. Altogether, 100 overflows, many of them at a very high setting, were classed as unsatisfactory for this reason alone and out of the total of 149 overflows subject to this effect, 118 were concentrated in two county boroughs with a combined population of nearly 800 000. It seems, therefore, that the incidence of groups of closely-spaced overflows is unlikely to be widespread but will be confined to a limited number of densely built-up communities.

53. There were 146 overflows (17 per cent of the total) at settings of less than 6 DWF and 111 of these were considered unsatisfactory, mostly because of "operation in dry weather" or "too frequent operation in wet weather"—as might have been expected. There

* The survey was carried out prior to the formation of the river authorities.

† Metric conversions from British units are given in Appendix 2.

Fig. 1. ANALYSIS OF OVERFLOWS IN RELATION TO SEWAGE AND STREAM FLOWS (NUMBERS AGAINST SYMBOLS INDICATE TOTALS).

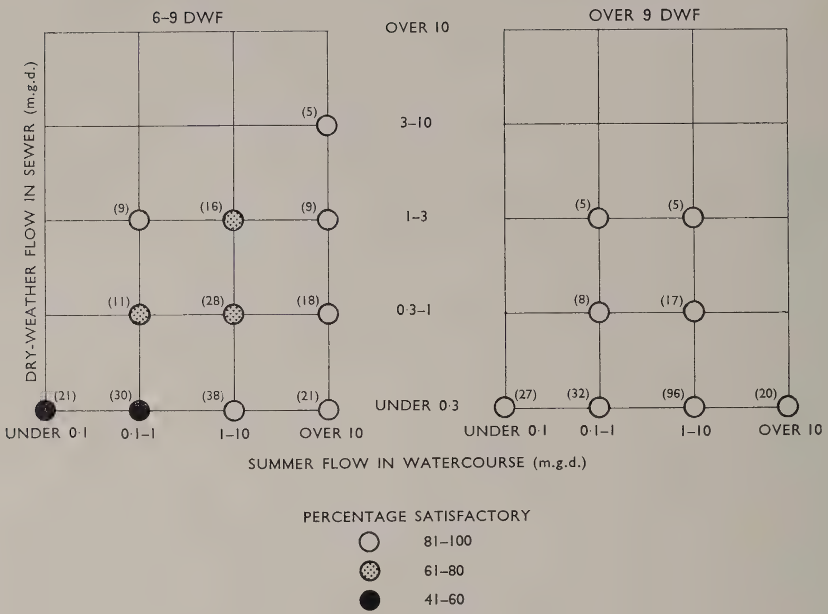
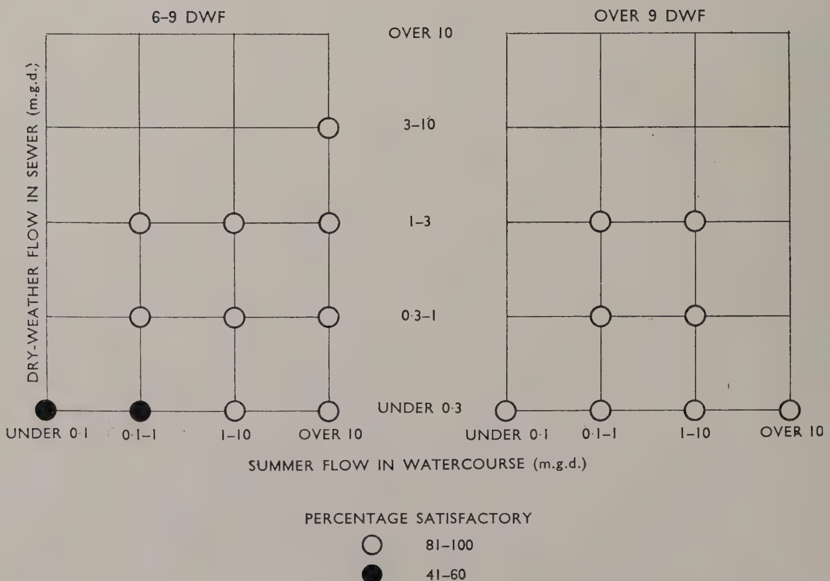


Fig. 2. REVISION OF FIG. 1 IF STRANDING OF SOLIDS ELIMINATED.



were also 21 overflows at settings of 6 DWF or higher which were considered unsatisfactory for "too frequent operation in wet weather".

54. If we exclude from our consideration all overflows set at less than 6 DWF, together with the overflows in Category E that were classified as unsatisfactory solely because of their combined group effect, we find that, out of a total of 603 overflows set at 6 DWF or higher, only 106 (less than 18 per cent) were considered unsatisfactory because of their individual effect on the stream. Furthermore, if 23 overflows set at 6 DWF or above which were considered unsatisfactory solely for the reason of stranding of solids were to be reclassified as satisfactory (on the assumption that effective means of retaining such solids in the sewer could be devised), the proportion of unsatisfactory overflows set at 6 DWF or higher would be reduced to less than 14 per cent.

55. An attempt was made to see whether there was any consistent relationship between the percentage of overflows considered satisfactory, the setting of the overflow, the dry-weather flow in the sewer and the normal summer flow in the watercourse. The results of this analysis are shown in Fig. 1, which was prepared from the information provided, but is limited to those unsatisfactory overflows in Categories A, B, F and G (unsatisfactory for their individual and observable effect) that were set at 6 DWF or higher and for which full particulars of sewage flow, setting and stream flow were provided. (Totals of less than five overflows are ignored.)

56. From the data selected in this way, it appears that the proportion of individually unsatisfactory overflows set at over 9 DWF is much smaller than that of overflows set at 6–9 DWF (the proportions were 6 and 23 per cent respectively). It can further be seen that overflows set at over 9 DWF are generally considered to be individually satisfactory whatever the flow in the receiving stream and whatever the dry-weather flow in the sewer, but that they are uncommon where the dry-weather flow exceeds 3 m.g.d. (In the whole survey there were only 7 such overflows.) Overflows set at 6–9 DWF appear to be generally individually satisfactory for sewage flows up to 10 m.g.d. when the stream flow exceeds 10 m.g.d., but when the sewage flow and the stream flow are both small a 6–9 DWF overflow has only about an even chance of being satisfactory. With moderate sewage flow and stream flow, and overflows set at 6–9 DWF, there is no consistent relationship. This suggests that other circumstances (such as location, accessibility, visibility, type of receiving stream and possibly strength of sewage) are of greater influence than the flows themselves and that, in certain cases, the proper siting of overflows is as important as their proper setting.

57. The hypothetical effect of installing efficient screens on the 23 overflows set at 6 DWF or higher, and considered unsatisfactory for stranding of solids only, is shown by Fig. 2 which is a variation of Fig. 1 by re-classifying the 23 overflows as satisfactory. The 6–9 DWF diagram is affected by this adjustment and it would seem from all the information that much of the trouble from individual overflows would be eliminated by settings between 6 DWF and 9 DWF coupled with the installation of efficient screens where necessary.

58. There seems little doubt that a main reason why many overflows are unsatisfactory is that the settings are too often below what has for decades been the normal standard, namely 6 DWF. We are also convinced that there are generally too many storm overflows and that there would be advantages in reducing their numbers. The excessive numbers usually occur on the older sewerage systems which have been developed progressively over very long periods, and in a lot of cases they cannot be operating in a co-ordinated manner.

59. Whilst there should be as few overflows as possible and sewer capacity should normally be used to the fullest extent, it is nevertheless better to overflow a weak domestic sewage than a strong industrial one, and an otherwise unnecessary overflow sited above the entry of a very toxic industrial effluent might with advantage be retained.

60. This leads us to suggest that there is scope for better management of sewerage systems which would lead to the optimum use of overflows and available sewer capacity, and we believe that much would be gained by close co-operation between sewerage authorities and river authorities in the study of this aspect and in the planning of remedial measures.

Reference

1. U.S. Department of Health, Education and Welfare. Pollutional Effects of Stormwater and Overflows from Combined Sewer Systems. A Preliminary Appraisal. (Public Health Service Publication No. 1246, 1964.)

CHAPTER 3. FIELD STUDIES ON FLOW AND COMPOSITION OF STORM SEWAGE

61. In order to collect information on the flow and composition of storm sewage, a programme of field studies was undertaken by the Water Pollution Research Laboratory. From some 60 sites, sewered on the combined system, suggested by river boards, three were selected, one at Northampton and two in Yorkshire—Brighouse and Bradford. The investigations took just over 5 years, flows of dry-weather and storm sewage being recorded from February 1960 to January 1962 at Northampton, from November 1958 to December 1961 at Brighouse, and from February 1961 to January 1964 at Bradford. At each site records of rainfall and the composition of storm sewage were maintained throughout most of the period and the composition of dry-weather sewage was measured on a number of occasions within the period. (Final samples of dry-weather sewage at Brighouse were taken in January 1962 after completion of the rest of the field work at that site.)

62. The confidence with which results of the field work may be assumed to be generally applicable depends largely on whether these three sites were typical of the country as a whole. We have not the information from which to decide how typical they were, but it is worthwhile examining the range of conditions covered by the three sites. Clearly, if the sites were similar in all important respects, the results could not confidently be assumed to apply to sites with very different characteristics. On the other hand, if they were very dissimilar and gave concordant results, then these results would be more likely to be generally applicable.

63. Some characteristics of the three drainage areas and sewerage systems are shown in Table 3. The Northampton area was developed in the latter half of the 19th century and was mainly residential. A distinctive feature of the sewerage system was that over three-quarters of the total sewer length of 9 miles consisted of brick sewers of 33 or 36 in × 24 in, and by present-day practice many of the head sewers would be considered unnecessarily large. The Brighouse area consisted largely of open spaces, and although the development was again mainly residential —most of the property was at least 100 years old— industrial effluent from four textile mills and a dye works formed a significant proportion of the dry-weather flow. With housing development and changes in the sewerage system, the population rose from 5200 to 6300 during the investigation, and the impermeable area from 59 to 68 acres; the figures quoted in Table 3 are weighted averages for the whole period. The sewers in this area were mainly salt-glazed ware, in sizes up to 24 in, with a few lengths of concrete sewer in larger sizes up to 36-in diameter. The Bradford area consisted almost entirely of a post-war housing estate, but there was, in addition, an institution (population about 605) lying inside the area served by the combined system and from which the sewage (but not the surface water) entered the sewerage system. The total contributory population was about 6150. The sewers were of glazed earthenware, in sizes ranging from 4 in to 18 in, with two lengths of concrete sewer, 21-in and 24-in diameter, immediately upstream of the overflow.

64. The range of variation covered by many of the factors in Table 3 is not very great, and when any one of them is likely to be of importance, caution must be exercised in applying the results to areas where the relevant factor is very different in magnitude.

TABLE 3. Characteristics of drainage areas and sewerage systems

Town or City Name of drainage area	Northampton St. Andrews	Brighouse Rastrick	Bradford Cooper Lane
Total area (acres)	229	594	167
Roofed or paved area (acres)	115	65	47
Roofed or paved area (per cent of total)	50	11	28
Population	9600	5800	5545*
Population density (persons/acre)	42	10	33
Impermeable area per person (yd^2)	58	54	41
Highest invert (ft above O.D.)	313	750	950
Total fall to lowest invert (ft)	87	434	153
Total length of sewers (miles)	8.9	10.6	6.6
Mean cross-sectional area of sewers (ft^2)	3.93	1.04	0.73
Median gradient of sewers	1 in 78	1 in 23	1 in 49
Time of concentration (min)	23	19	12
Average dry-weather flow (m.g.d.)	0.323	0.56	0.179
Lowest rainfall total for 12 consecutive months of investigation (in)	18.6	26.8	22.7
Highest rainfall total for 12 consecutive months of investigation (in)	33	38.3	33.7
Standard average rainfall (in/year)	24.6	32†	32†

* Total contributory population 6150.
† Estimated from neighbouring stations.

65. The survey of existing discharges, discussed in Chapter 2, yielded some information on three of the factors listed. For a total of 849 overflows, the average contributing population per overflow was about 4000. This is below the range covered by the field study, but with overflows in series—and it was estimated that, on average, sewage passed 4 overflows before reaching the treatment works—the average population contributing to each overflow must be considerably greater than this. The survey also provided information on the proportion of overflows on sewers having dry-weather flows within given ranges; examination of these data suggested that at roughly 50 per cent of these overflows the dry-weather flow fell below the range covered by the study (0.18 to 0.56 m.g.d.) and in 30 per cent it was above. The survey covered the areas of 52 local authorities; for 56 per cent of these areas the standard average rainfall lay within the range 24.6–32 in/year, and in only 5 of the areas did it lie outside 18.6–38.3 in/year—the range observed during the experimental work at the three sites.

Site works and instrumentation

Northampton

66. There was no overflow at the Northampton site, but it was possible to calculate the flow and composition which would have been discharged had an overflow existed, assuming that the flow to treatment remained steady at the first-spill value and that the composition of the overflowed and treated storm sewage was the same. A stilling chamber and compound flume were constructed at the lowest point in the drainage area, and flows were determined by means of a level recorder with a chart speed of 6 in/h. Mounted over the stilling chamber was an automatic sampler, consisting essentially of a scoop which delivered a one-pint sample into a receiving bottle; 36 such bottles in a circular rack were filled in turn. Because the scoop was drained after each operation, samples were not significantly contaminated by residual portions of previous ones. Operation of the sampler was controlled by a timer unit, activated only when the water level was above the end of a pre-set electrode.

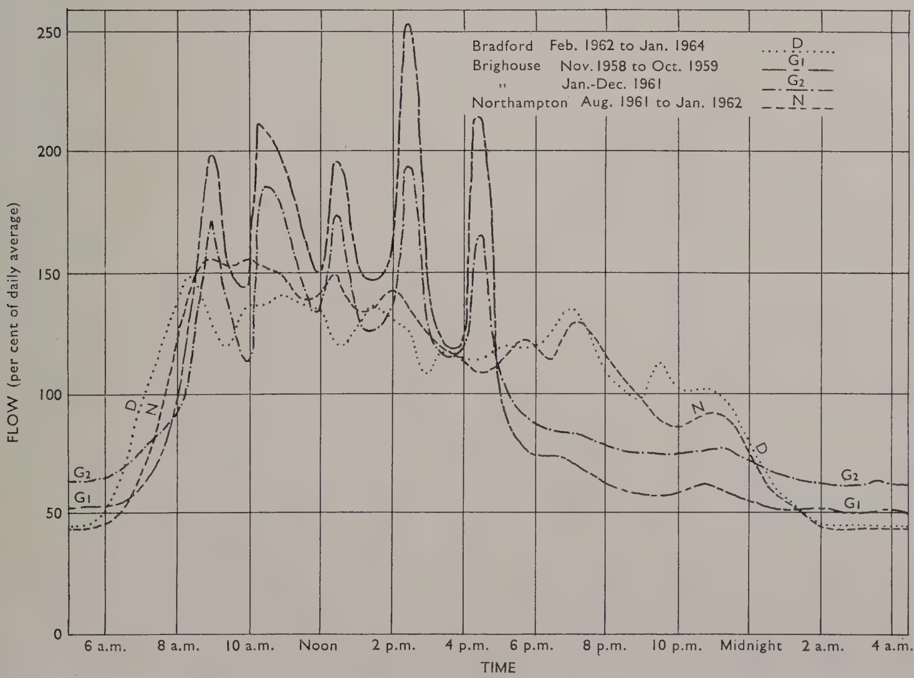
67. Two electrically-operated rainfall recorders, each with a chart speed of 6 in/h, were installed in the drainage area; one of these gave a direct measurement of rainfall intensity as well as of cumulative rainfall. The arithmetic mean of the records from both instruments was taken as the rainfall over the whole area.

Brighouse

68. The Brighouse overflow, which was fed by a 36-in sewer, was a double-sided weir $33\frac{1}{2}$ ft long (total weir length 67 ft) with its crests about 8 in above the outgoing invert. It tapered from 42 to 24 in with a final contraction to the 18-in downstream sewer. A metal plate was fitted across the entrance to the downstream sewer at sill level to prevent surcharging and this necessarily altered the performance of the overflow.

69. Hydraulic and site conditions prohibited the constructing of stilling chambers and flumes for flow measurement; rates of flow were therefore determined from the depth of flow in the sewers. Flow

Fig. 3. DIURNAL VARIATIONS IN DRY-WEATHER FLOW AT BRADFORD, BRIGHOUSE AND NORTHAMPTON.



recorders were installed downstream of the overflow both on the foul and the storm sewers. These were calibrated by means of radiochemical-dilution and salt-velocity experiments, but considerable difficulty was experienced and it had to be accepted that measurements of dry-weather flow might be in error by as much as 25 per cent. An automatic sampler operated in the storm sewer. Two total-rainfall recorders and one combined total-rainfall and rainfall-intensity recorder (installed by the Road Research Laboratory for research on urban sewer systems) were made available for the investigations; the average rainfall over the whole area was calculated by Theissen's Method[1].

Bradford

70. At the Bradford site, sewage entered the overflow chamber from a 6-ft length of 24-in half-round channel, at the upstream end of which there was a manhole, 6 ft long, receiving sewage from a 12-in sewer (at a gradient of 1 in 22) running in line with the axis of the overflow and a 24-in sewer (at 1 in 15) at about 45° to the axis. The overflow was a double-sided weir 20 ft long (total weir length 40 ft), the internal width tapering from about 32 to 20 in; the final few feet were further tapered to connect with the outgoing 12-in foul sewer. The weir crests were set about 6½ in above the invert of the trough.

71. Stilling chambers and measuring flumes were constructed on both the foul sewer and the storm sewer downstream of the overflow, and flows in both sewers were measured by depth recorders. An automatic sampler was mounted above the stilling chamber. Two total-rainfall recorders and one combined total-rainfall and rainfall-intensity recorder were installed in the area; the average rainfall over the whole area was calculated as at Brighouse.

Rainfall

72. The yearly rainfall totals recorded during the period of study at each site are shown in Table 4.

TABLE 4. Yearly rainfall totals (inches) during period of investigation at three experimental sites

	Northampton	Brighouse	Bradford
1st year	32.22	24.75	33.67
2nd year	19.38	36.62	23.92
3rd year	—	31.80	27.49
Range of monthly totals	0.12–5.27	0.09–5.28	0.56–4.95
Standard average (1916–1950)	24.56	32*	32*

* Estimated from neighbouring stations.

73. Examination of the frequency distributions of the monthly totals—expressed as a percentage of the overall average monthly total for the station so as to make allowance for differences in average rainfall between the three stations—provided no evidence to suggest that the general pattern of rainfall on the three drainage areas was markedly different.

Dry-weather flow

74. The flow was read from the charts at half-hourly intervals for the whole period of the investigation at each site and, after excluding the data for periods when rainfall was influencing the flow, monthly averages of the flow at each half hour of the day were calculated for weekdays, Saturdays and Sundays. At Northampton the excess flow attributable to rain generally appeared to be negligible within 2 h of the rain ceasing, but after heavy storms at Bradford it occasionally took over a day for the flow to settle down.

Diurnal variations

75. Diurnal variations in dry-weather flow on weekdays expressed as percentages of the 24-h average are shown in Fig. 3. At Brighouse, the data indicated a progressive increase in dry-weather flow throughout the investigations, and so curves are shown for two different periods. The unusual shape of these curves was caused by periodic discharges of industrial effluents between 9 a.m. and 5 p.m. Such circumstances are not typical of the country as a whole.

Average dry-weather flow

76. Table 5 shows the estimated average dry-weather flow for the whole period of the investigations at each site. The figures suggest that at Northampton and Bradford the dry-weather flow consisted mainly of domestic sewage, but that at Brighouse most of the flow must have been due to trade discharges and/or infiltration of ground water. At each site the dry-weather flow in winter (November–April) was significantly higher than in summer (May–October).

TABLE 5. Average dry-weather flow, based on 2–3 years' records for each site

Average dry-weather flow, 7-day week basis	North-ampton	Brighouse	Bradford
Whole year (m.g.d.)	0.323	0.56	0.179
Whole year (g.h.d.)	33.6	97	29.1
Winter (m.g.d.)	0.340	0.61	0.193
Summer (m.g.d.)	0.307	0.51	0.165
Winter minus summer			
(m.g.d.)	0.033	0.10	0.028
(per cent of average)	10	18	16
(g.h.d.)	3.4	17	4.6

77. Taking account of the average metered water consumption of industrial concerns in the Brighouse drainage area, and other available information, it was concluded that the average dry-weather flow at Brighouse was roughly made up of 0.16 m.g.d. (29 gal per head per day) of domestic sewage, 0.11 m.g.d. (18 g.h.d.) of industrial discharges, and 0.29 m.g.d. (50 g.h.d.) of infiltration water.

Flow in wet weather

78. The percentage run-off was calculated for all falls of rain which had a measurable effect on the sewage flow. The frequency of occurrence and the duration of flows in excess of particular values were also noted. From these results it is possible to estimate how often and for how long an overflow,

constructed so as to limit the flow to treatment to a given value, would have operated at each site, and to calculate the volume of storm sewage which would have been discharged.

79. It was hoped that if consistent results were obtained at each site throughout the period of the observations, the results might justifiably be used to predict future conditions at these sites, and that if a consistent relationship were found to be applicable to all three sites, it might be possible to use this in the prediction of overflow performance in other areas.

Percentage run-off

80. The volume of run-off was found for individual periods of rainfall by subtracting from the measured flow the average dry-weather flow appropriate to the site, the month, the day of the week (weekday, Saturday, or Sunday), and the time of day, and graphically integrating this excess flow with respect to time. The resulting volume can be expressed as a percentage of the volume of rain falling on the whole drainage area, but because most of the surface water entering the sewers will come from the rain falling on the impermeable areas connected to the sewers, it is more convenient when comparing the results from the three sites to take account of the impermeable area only. In this report, the percentage run-off is defined as the volume of run-off observed to pass the gauging point, expressed as a percentage of the volume of rain falling on the impermeable area (roofed plus paved) drained by the system. In averaging the values for a number of falls of rain, or a number of months, the mean value of percentage run-off was derived from the relevant total volumes of run-off and rainfall; combined averages over the three drainage areas were weighted according to the rainfall in each area.

81. At Bradford anomalous results were found for a number of heavy storms, the "run-off" continuing for far longer than would be expected from an area of this size, and exceeding the total volume of rain falling on the impermeable parts of the drainage area and grassed areas such as verges. On one occasion it continued for more than 2 days after the rain had stopped, and the total run-off was estimated to have been equivalent to 100 per cent of the rainfall on the whole drainage area, or 360 per cent of that on the impermeable area. The source of this water was never found, but it is suspected that it may have entered from an underground stream.

82. Excluding the Bradford data relating to the comparatively few occasions when such prolonged run-off occurred, average values for the percentage run-off for the whole of the experimental period at each site are given in Table 6. The yearly average run-off from each area was close to three-quarters of the rain falling on the impermeable area, but in view of the variations in the individual monthly averages (from 36 to 116 per cent) the closeness of the agreement between yearly averages must be to some extent fortuitous. Nevertheless, the agreement found between such dissimilar areas as Northampton and Brighouse is of interest. Because of the exclusion of some of the data, the Bradford figures other than those for April, May and June cannot be taken as reliable averages.

TABLE 6. Average values of percentage run-off based on impermeable area. Data for periods of prolonged run-off at Bradford excluded

	Northampton	Brighouse	Bradford	Average
January	84	77	96	82
February	83	36	60	71
March	63	37	116	56
April	74	58	65	67
May	72	49	63	63
June	65	55	63	64
July	64	74	75	70
August	66	69	58	66
September	84	86	79	83
October	79	87	86	82
November	90	103	83	94
December	90	86	86	89
Summer	72	76	72	73
Winter	84	76	83	81
Year	77	76	74	76

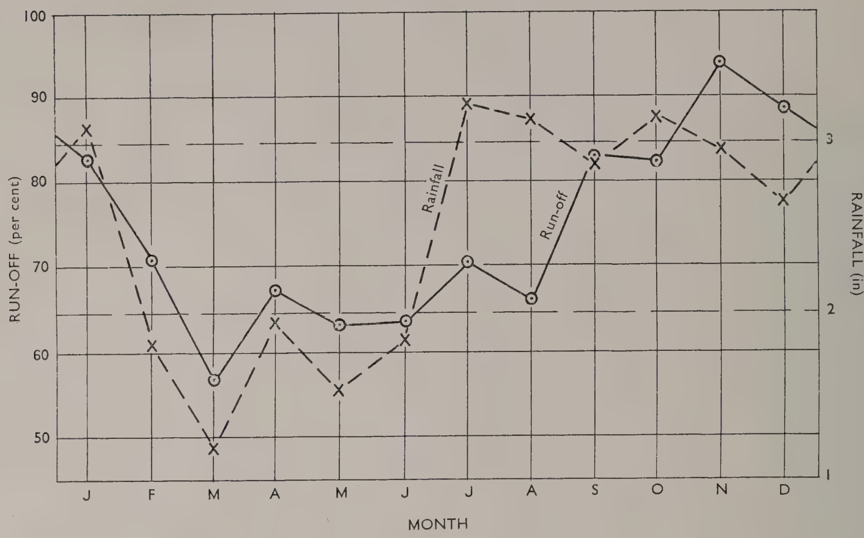
83. The combined monthly average data for the three sites are shown by the encircled points in Fig. 4; the crosses show the average monthly rainfall totals during the months used in obtaining the run-off, an average of 2 years' data being used for each month of the year at each site. The seasonal variations are largely associated with the wetness of the months. With the exception of the two warmest months of the year (July and August) there was a roughly linear relation between the monthly averages of percentage run-off and rainfall.

84. The relationship between rainfall and run-off was examined in rather more detail[2] for the data covering a period of 19 months at the Northampton site. It was found that the percentage run-off tended to increase with rainfall intensity, the intensity taken being the highest average intensity over any period of 23 min in each storm, 23 min being the time of concentration. At intensities of around 0.03 in/h the average run-off was about 50 per cent, whereas at 0.3 in/h it was 80 per cent. For a given intensity, the percentage run-off in winter (November–April) was significantly higher than that in summer (May–October), the average difference being equivalent to a run-off of 12 per cent. When no rain had fallen over the previous 24 hours, the percentage run-off was less than average. It is of interest to note that the percentage run-off found from the total run-off and total rainfall showed good agreement with the value calculated from the peak flow and the average rainfall intensity over the 23-min period when this was greatest.

Duration of flows in excess of particular values

85. The times during which any particular flow was exceeded were examined for the whole of the experimental period for all three sites. To enable comparisons to be made between data for areas having different

Fig. 4. RELATION BETWEEN AVERAGED MONTHLY VALUES OF PERCENTAGE RUN-OFF (based on impermeable areas) FOR THREE SITES AND CORRESPONDING MONTHLY RAINFALL TOTALS.



rainfall totals, or between data for the same area but for different periods, durations were expressed in terms of hours per inch of rainfall (D h/in), on the assumption that the total time during which a given flow is exceeded is proportional to the total rainfall during the period for which records are being considered. This is an approximation only, and cannot apply to flows within the dry-weather range, but it is reasonable to assume that, for a particular area, during two years of equal rainfall, the total time a given flow is exceeded will be roughly twice that during either year, and that the greater the total rainfall in a year the greater will be the time that a given flow is exceeded. The examination of data from the individual drainage areas showed that results for different periods were similar when allowance was made in this way for differences in rainfall.

86. In order to compare the results from different drainage areas it was necessary to express storm flows as the excess flow (actual flow minus dry-weather flow) divided by the impermeable area, and the most rational units for such an expression appeared to be in/h (1 in/h=0.543 m.g.d./acre). This is then the steady rainfall intensity which (with 100 per cent run-off from the impermeable area and none from the permeable), when combined with the normal average dry-weather flow, would produce the actual flow in the sewer. Thus, for a flow of Q m.g.d. in a sewer normally carrying a dry-weather flow of q m.g.d., draining an impermeable area of A acres, the steady rainfall intensity i in/h would be given by

$$i = 1.84\,(Q-q)/A \text{ in/h.*} \qquad (1)$$

87. The actual rainfall required to raise the flow by this amount would, of course, normally be greater than i in/h, even if the intensity remained steady during the time of concentration, since the total run-off is generally less than 100 per cent of the rain falling on the impermeable area. (It may be noted that in these units i is numerically almost identical with the excess flow in terms of cusecs per impermeable acre.)

88. In Fig. 5(a) the distributions of duration are shown for each of the three drainage areas for values of i up to 0.2 in/h; two curves, A and B, are shown for Bradford, these relating to the data including and excluding periods of prolonged run-off respectively. However, it is not known whether, had there been no anomalous periods of prolonged run-off, the curve for Bradford would have lain between Curves A and B, and the Bradford results should not therefore be presumed to support those from the other two sites. The curves for Northampton and Brighouse are very close together and, over most of the range covered by the diagram, lie between the Bradford curves. The smooth continuous curve satisfactorily fits the Northampton and Brighouse data over the range of flows equivalent to 3–40 times the average flow in dry weather at Northampton and 2–16 times at Brighouse; this curve, which was fitted empirically to the data from these two sites, gives the duration, D hours per inch of rainfall, according to the equation

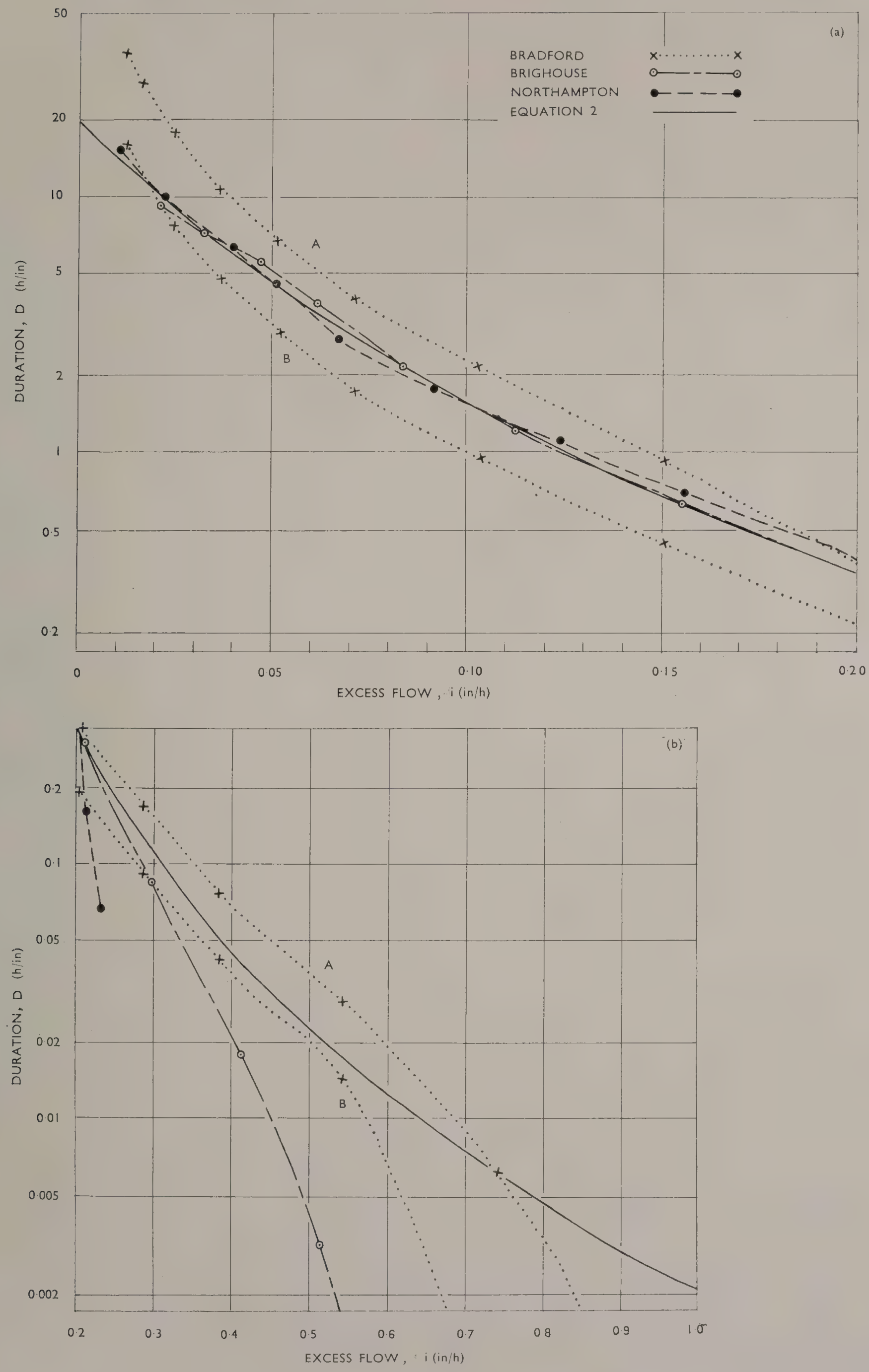
$$D = 20\,(1+8.8i)^{-4} \text{ h/in.} \qquad (2)$$

89. The distributions for values of i greater than 0.2 in/h are shown in Fig. 5(b). The curves are widely different—due in part to the relative infrequency of

* Numerical conversion factors to express the equations in metric units are given in Appendix 2.

Fig. 5. AVERAGE DURATION OF FLOWS IN EXCESS OF PARTICULAR VALUES, PER INCH OF RAINFALL, USING 3, 3 AND 2 YEARS' DATA FOR BRADFORD, BRIGHOUSE AND NORTHAMPTON RESPECTIVELY.

CURVES A and B (FOR BRADFORD) RESPECTIVELY INCLUDE AND EXCLUDE DATA FOR PERIODS OF PROLONGED RUN-OFF.



high flows (and hence to insufficient data to be considered representative) and in part to peculiarities of the individual systems[2]. The continuous curve given by Equation 2 is included in Fig. 5(b) but the equation was derived from the data of Fig. 5(a) only.

90. It should perhaps be mentioned that although Fig. 5 shows a fair measure of agreement between the yearly distributions of duration at two of the sites, and although there was in general good agreement between different years at each site (a published paper[3] gives comparison of two consecutive years at Northampton), there were large differences between the summer and winter distributions, so that if summer conditions only were being considered, Equation 2 would not be relevant. The ratio of winter to summer durations was found to fall from $1\frac{3}{4}$ at $i=0.01$ in/h to zero at a value of i rather more than 0.3 in/h. Such results must be attributed to differences between summer and winter rainfall patterns, intense storms being more common in the summer.

91. If it were required to stipulate overflow settings in terms of permissible total duration of overflow, T hours per year, in an area where the standard average rainfall was R inches per year, then Equation 2 could be multiplied by R and re-arranged to give

$$i=0.1135\left\{\left(\frac{20R}{T}\right)^{\frac{1}{4}}-1\right\} \text{ in/h.} \qquad (3)$$

The term i is the excess flow (expressed as an equivalent intensity of rainfall on the impermeable area) to be retained before spill, and may be converted to overflow setting (Q m.g.d.) by application of Equation 1. Thus

$$Q=q+0.0617\ A\left\{\left(\frac{20R}{T}\right)^{\frac{1}{4}}-1\right\} \text{ m.g.d.} \qquad (4)$$

92. If the overflow setting is "n DWF", then $Q-q=(n-1)q$, and Equation 4 can be re-arranged as

$$T=20R\left\{1+16.2(n-1)\frac{q}{A}\right\}^{-4} \text{ hours per year.} \qquad (5)$$

If the dry-weather flow is w g.h.d. and the impermeable area is a yd²/head, the term $16.2q/A$ may be replaced by $0.078\ w/a$, and T may then be evaluated on this alternative basis.

93. As an illustration, Table 7 shows the results of applying the modified equation to a drainage area where the annual rainfall is 30 inches and the impermeable area per person is 50 yd², for various dry-weather flows and overflow settings.

TABLE 7. Total annual duration of discharge (hours) from hypothetical overflow in a drainage area with annual rainfall of 30 inches and impermeable area of 50 yd² per person

Dry-weather flow (g.h.d.)	Overflow setting (multiples of DWF)		
	6	9	12
30	259	168	114
40	202	119	74
50	140	86	50
60	129	64	35

94. Before the start of the field studies the Laboratory looked at the flow records for a partially-separate system in the Borough of Luton, to see how such data might be treated and what difficulties might be expected to arise in the subsequent investigations. The records for 1954 and 1956 were selected for detailed examination and these subsequently presented an excellent opportunity of testing the application of Equation 2.

95. The points plotted in Fig. 6 show the average yearly durations found from these records, the flow being expressed in terms of i (as given by Equation 1) and thus involving the observed flows (Q m.g.d.), the average dry-weather flow ($q=0.53$ m.g.d.), and the total impermeable area connected to the sewer ($A=67$ acres)—though this last term is not required for the scale at the top of the diagram. The curve in the same diagram is the distribution calculated from Equation 2, multiplied by the average annual rainfall during the two years ($R=26.4$ in) to express the durations as yearly totals. The agreement between the two is remarkably close—even well beyond the limit of $i=0.2$ in/h used in deriving the equation.

96. The total area served by this partially-separate sewerage system was 840 acres, of which the impermeable area draining to the sewers accounted for only 8 per cent. The contributory population was 13 600 and the time of concentration about 40 min. In Table 3 is shown the range of conditions covered at the three experimental sites. Inclusion of the Luton site increases the range of three of these: that of populations becomes 5800 to 13 600, and that of impermeable areas per person becomes 24 to 58 yd², with times of concentration of between 12 and 40 min.

97. The Luton data thus provide striking confirmation of Equation 2 and must therefore add greatly to the confidence with which it may be applied to other areas of the same general size.

Volume discharged from hypothetical overflow

98. Integration of time with respect to flow gives a volume, and integration of D h/in with respect to i in/h, from any particular value of i to infinity, gives a dimensionless quantity, v, which is the ratio of the volume that would be discharged from an overflow set at this value of i to the total volume of rain falling on the impermeable area (provided the flow to treatment is restricted to that at which the overflow operates). In practice, in the Brighouse and Bradford systems, the flow to treatment increased with increasing discharge from the overflow, and the volume discharged as storm sewage was substantially less than from the hypothetical overflow. The operation of these two overflows will be considered later. There was, of course, no overflow on the area studied at Northampton.

99. The volumes found by graphical integration of the curves in Fig. 5 are shown in Fig. 7. It will be remembered that at intensities higher than 0.2 in/h the curve described by Equation 2 lies well above the observed data (possible reasons for this have already been given)

Fig. 6. AVERAGE YEARLY DURATION OF FLOWS IN EXCESS OF PARTICULAR VALUES, USING DATA FROM A PARTIALLY-SEPARATE SYSTEM AT LUTON, 1954 AND 1956. PLOTTED POINTS SHOW OBSERVED VALUES; CURVE OBTAINED FROM EQUATION 2.

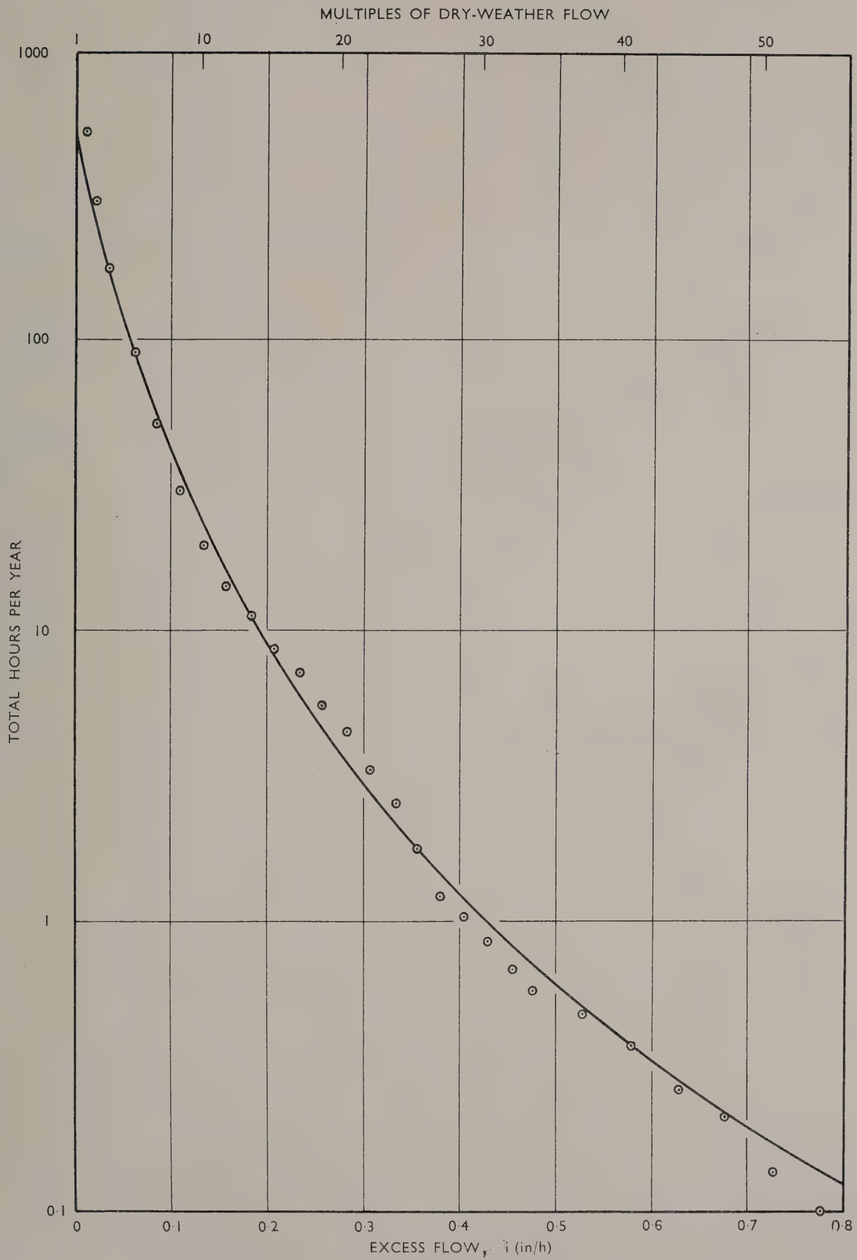
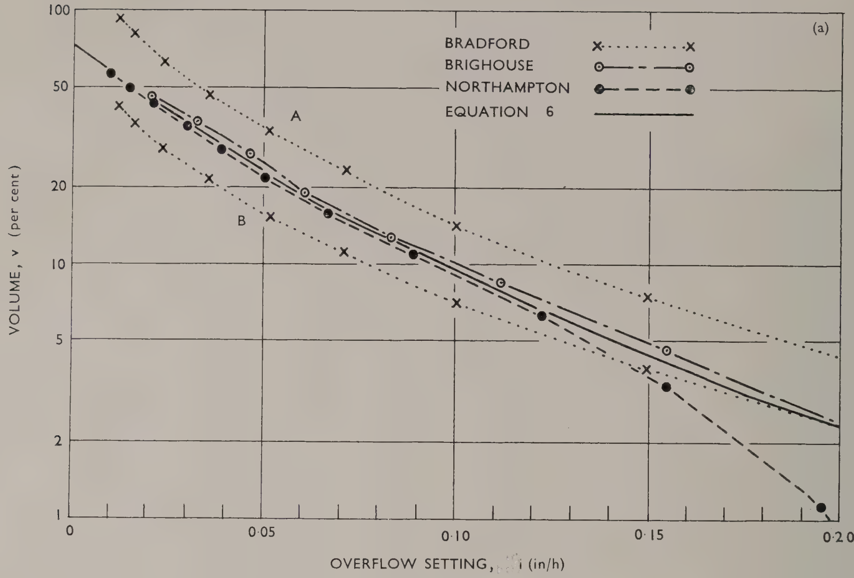
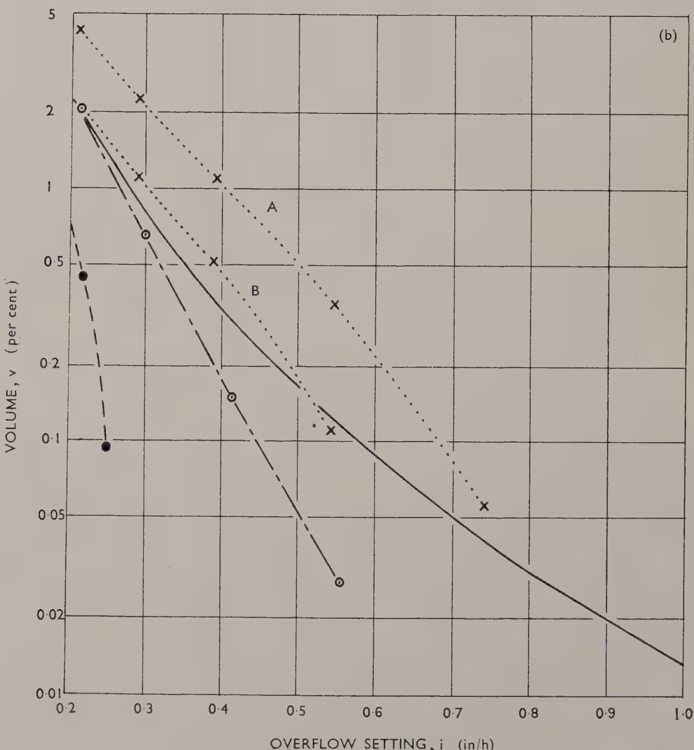


Fig. 7. ESTIMATED AVERAGE VOLUME DISCHARGED FROM HYPOTHETICAL OVERFLOW (FOUND BY INTEGRATION OF CURVES IN Fig. 5) EXPRESSED AS PERCENTAGE OF TOTAL RAINFALL ON IMPERMEABLE AREA. "A" AND "B" AS IN Fig. 5.





and it is clear that integration of this curve from $i=0.2$ in/h to $i=\infty$ must give too high a value for v at $i \geqslant 0.2$ in/h, which will correspondingly affect the distribution at lower values of i. Consequently, it is more satisfactory to fit a separate curve to the data of Fig. 7; the one shown is for

$$v=75\ (1+6i)^{-4.4}\ \text{per cent.} \qquad (6)$$

When i is put equal to zero, the volume discharged is the whole of the run-off: the value of 75 per cent given by the equation is within 1 per cent of the average percentage run-off from the impermeable area at the three sites.

100. The results for the drainage area at Luton are compared in Fig. 8 with those given by Equation 6. For flows from 3 to 12 DWF the percentage discrepancy between the two sets of values increases from 10 to 30; for all flows up to 50 DWF the discrepancy never exceeds a factor of 2. This is very small when compared with the 1000-fold range in predicted values.

Frequency of occurrence of particular flows

101. Information about the number of occasions on which particular flows are exceeded makes it possible to estimate the frequency of operation of an overflow at any particular setting. However, in a single day there may be several storms during each of which the overflow operates more than once, and as far as the effect on a receiving watercourse is concerned the total number of discharges is probably of little interest. Of more practical importance are the number of separate run-off periods, and the number of days, when discharge occurs. (A period of run-off has been arbitrarily defined as one in which rainfall causes the flow to rise by more than 0.1 m.g.d. above the dry-weather flow appropriate to the drainage area, the time of day, the day of the week, and the month.)

102. The number of periods of run-off in which different flows were exceeded, during the whole of the periods for which the data were examined, is shown for each site in Fig. 9(a) where the results have been expressed in terms of frequency per inch of rainfall. The Curve A for Bradford lies below Curve B for values of i up to about 0.25 in/h, although in Figs. 5 and 7 Curve A was the higher; this is because the relatively few storms included in Curve A and excluded from Curve B were associated with large falls of rain and, had the run-off not been unduly prolonged, would have given rise to a considerably larger number of periods of run-off.

103. Over the range of excess flows of practical interest in determining the settings of overflows, the data are reasonably consistent. For example, the number of periods of run-off per inch of rainfall shown for $i=0.027$ in/h (equivalent to an overflow set at 6 DWF on a domestic sewerage system where the water consumption is 30 gal per head per day and the impermeable area 50 yd²/head) is 5.4 using the Northampton data, 5.3 using Brighouse, and 4.8 and 7.7 using Bradford Curve A and Curve B respectively. Thus for a yearly average rainfall of 30 inches these

Fig. 8. AVERAGE VOLUME OF EXCESS FLOW DUE TO RAINFALL, DURING TWO YEARS AT A SITE IN LUTON, AS PERCENTAGE OF TOTAL RAINFALL ON IMPERMEABLE AREA. PLOTTED POINTS SHOW OBSERVED VALUES; CURVE OBTAINED FROM EQUATION 6.

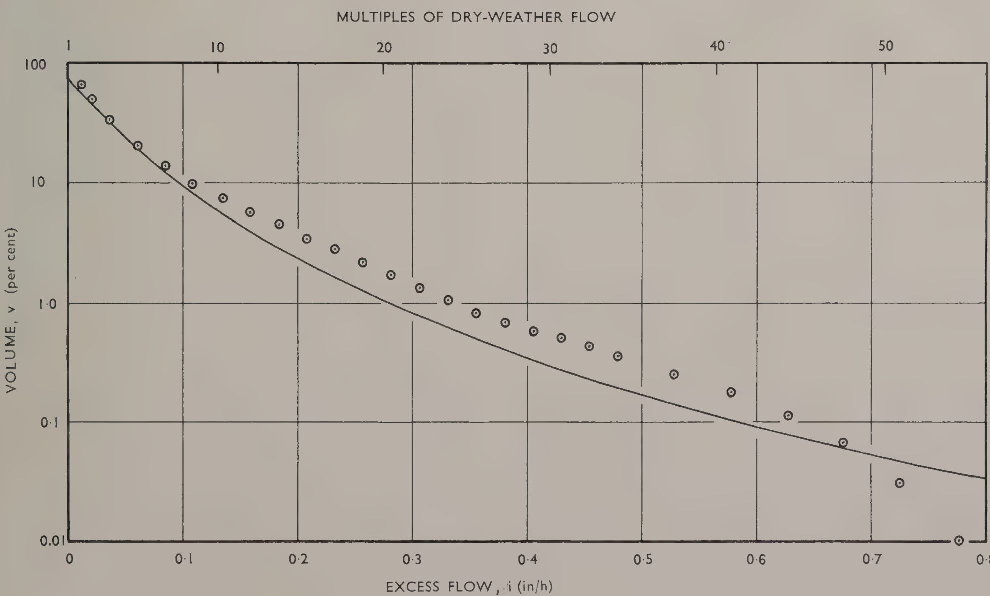
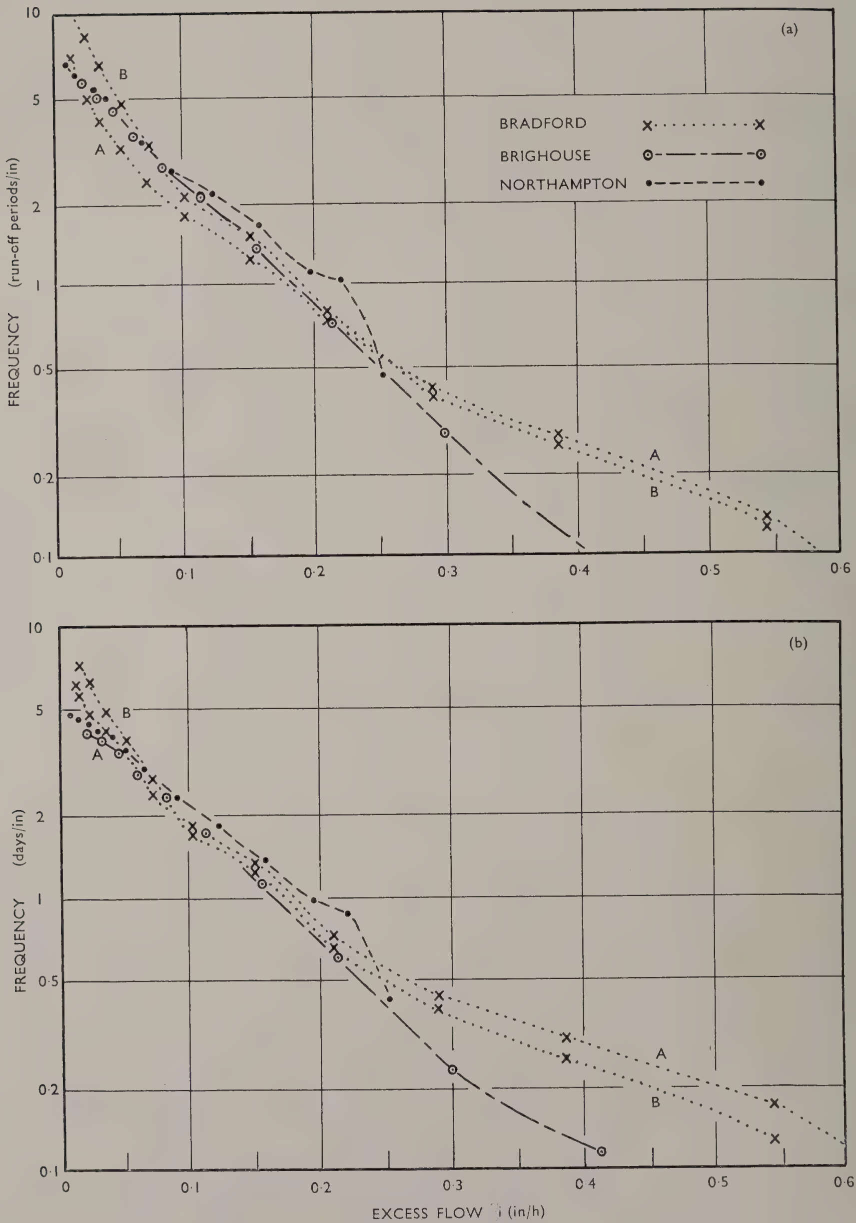


Fig. 9. AVERAGE NUMBER OF (a) PERIODS OF RUN-OFF AND (b) DAYS WHEN FLOW EXCEEDED PARTICULAR VALUES PER INCH OF RAINFALL. "A" AND "B" AS IN Fig. 5.



figures give the number of run-off periods during which discharges would be expected to occur as about 160 per year from both the Northampton and Brighouse data, and roughly 145 and 230 from the two sets of data from Bradford. Of the two Bradford values, the former is probably the more relevant, as all the large storms could have produced flows in excess of 0.027 in/h even in the absence of prolonged run-off, but will be too low because some periods of run-off which would otherwise have been separate will have been joined together. It would probably therefore be reasonable to take 160 periods per year—the value found from the data for Northampton and Brighouse—as the average for all three sites at this setting.

104. The results in terms of number of days on which discharge would have occurred are shown in Fig. 9(b). At 0.027 in/h the curves show discharges on 4.2, 4.0, and 4.6 days/in respectively for Northampton, Brighouse, and Bradford A, so that, on average, discharges might be expected on 126, 120 and 138 days/year for a rainfall of 30 in/year.

105. From Equation 2, an overflow set at 0.027 in/h in an area with an annual rainfall of 30 in would be expected to discharge for about 260 hours during the year; using the values given in the previous two paragraphs, this would give about $1\frac{1}{2}$ h as the average period of each discharge and about 2 h as the average discharge per day of operation.

Overflow settings

106. By replacing the term Q in Equation 1 by a suitable range of overflow settings (in m.g.d.), and applying the resulting values of i to Fig. 9 and Equations 2 and 6, an estimate of the effect of changes in overflow setting on the frequency, duration, and volume of discharge from a hypothetical overflow can be obtained. Table 8 has been calculated in this way for a domestic drainage area where the dry-weather flow is equivalent to 30 gal/head per day, the impermeable area 50 yd^2/head, and the annual rainfall 30 in. In order to make it easier to compare these results with current practice, overflow settings have been expressed as multiples of the average dry-weather flow. It is seen that raising the setting from 6 to 8 DWF (a 40 per cent increase in the flow in excess of the dry-weather flow) would be expected to reduce the frequency of operation of the overflow by less than 10 per cent, the total duration of discharge by 25 per cent, and the volume discharged by 21 per cent. It may be noted that over the range of settings covered by Table 8 the volume discharged is almost inversely proportional to the setting.

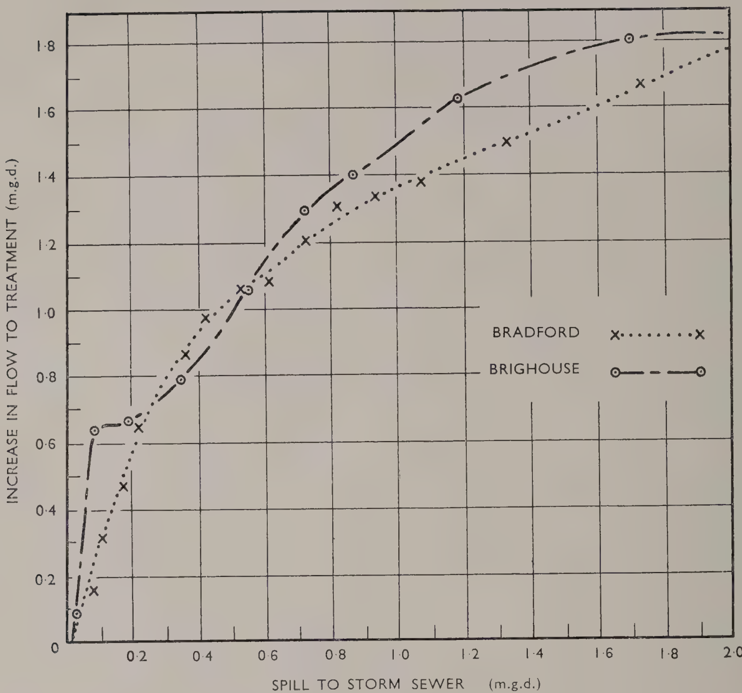
Table 8. Effect of changes in setting on discharge from hypothetical overflow in a drainage area with annual rainfall of 30 inches, impermeable area of 50 yd^2 per person, and dry-weather flow of 30 g.h.d.

Overflow setting (DWF)	6	8	12	20
Excess flow (g.h.d.)	150	210	330	570
No. of discharges per year	161	149	115	72
Total duration of discharge (h/year)	259	193	114	47
Volume discharged (per cent of rainfall on impermeable area)	39	31	20	9

107. Discussion has so far been confined to the performance of hypothetical overflows for which the flow at initial discharge is the maximum flow passed to treatment. The overflows studied at Brighouse and Bradford did not conform to such ideal conditions, although, as is shown later in the report (Chapters 4 and 5), a much closer approximation could be readily obtained by a suitably designed overflow. At both sites the initial discharge probably resulted from splashing at the downstream end of the overflow trough; at Brighouse, further splashing was probably caused by sewage hitting the metal plate which cut down the foul-sewer orifice to sill level. At Bradford, the sinuous flow caused by the confluence of two sewers just upstream of the overflow chamber resulted in some spillage at one or two points along the trough before the main discharge started; theoretical calculations indicated that the flow at which first spill should occur at this site might vary by over 30 per cent depending upon the relative flows from the two incoming sewers. At both sites, after the main discharge had started, the flow to treatment continued to rise appreciably.

108. Because at each site the flows in the foul and storm sewers were recorded on different instruments the clock mechanisms of which did not always keep perfect time, and because the recorders on both sewers were some distance downstream of the overflows, it is not possible to determine accurately the flows at which these overflows operated; indeed, as will be apparent from the preceding paragraph, it is even difficult to define operation of overflows of these types. However, detailed examination of all the available data suggests that the first discharge occurred at about 3.5 m.g.d. at Brighouse and 1.6 m.g.d. at Bradford. Fig. 10 shows how the flow to treatment increased as the discharge from the overflows increased to 2 m.g.d., each curve having been derived from roughly a thousand pairs of values. It can be seen that when the spill at Bradford was 1.6 m.g.d., the flow to treatment had risen by 1.6 m.g.d.—to twice the flow at which first spill occurred. At Brighouse, although first spill occurred at about 3.5 m.g.d., there was probably no significant flow in the storm sewer until the flow to treatment had risen by 0.6 m.g.d., and until the spill reached 1.8 m.g.d. it was exceeded by the increase in flow to treatment. When the spill from this overflow approached 20 m.g.d. the flow to treatment rose to roughly 6.6 m.g.d.—nearly twice the estimated setting of 3.5 m.g.d.—despite the presence of the plate restricting the flow to the foul sewer. (Surcharging of this sewer probably occurred at about 10 m.g.d. before the plate was fitted.) It will therefore be appreciated that the custom of referring to the flow at which discharge first occurs as the setting of the overflow is misleading so far as overflows of this type (low side-weirs) are

Fig. 10. RELATION BETWEEN FLOWS SPILLED TO STORM SEWER (up to 2 m.g.d.), AT BRADFORD AND BRIGHOUSE AND INCREASE IN FLOW TO TREATMENT ABOVE THAT AT WHICH OVERFLOW OPERATES.



concerned. It would appear more sensible to express the setting of the Brighouse overflow as 3.5–6.6 m.g.d., the former figure indicating the flow at which initial discharge occurs and the latter the maximum flow to treatment; the setting of the Bradford overflow might be similarly given as 1.6–5 m.g.d.

109. When comparing flows at which discharge starts at different overflows it has been the practice to express them as multiples of the average dry-weather flow. Thus 1.6 m.g.d. at Bradford is equivalent to 8.8 DWF and 3.5 m.g.d. at Brighouse to 6.2 DWF. However, it was estimated that the dry-weather flow at Brighouse consisted of 0.29 m.g.d. of infiltration water, 0.11 m.g.d. of industrial effluent, and only 0.16 m.g.d. of domestic sewage; the flow of 3.5 m.g.d. is therefore equivalent to 13 times the dry-weather flow of sewage and industrial effluent, and to about 20 times that of domestic sewage alone.

110. During the period for which complete records were obtained, the total volume discharged from the Brighouse overflow was 6.6 per cent of the rainfall on the impermeable area. From the curves shown for Brighouse in Fig. 7 it can be seen that this percentage would have been discharged from an overflow set at i=0.131 in/h if this had been the maximum excess flow passed to treatment. (At this setting Equation 6 gives a discharge of 5.9 per cent.) Similarly, it is estimated that the volume actually discharged from the Bradford overflow was equivalent to that which would have been discharged by an overflow which restricted the excess flow passed to treatment to i=0.105 in/h.

111. The various ways of expressing the settings of these overflows are summarized in Table 9. The importance of designing overflows to control accurately the flow to treatment is illustrated by the performance of these two examples. Had these overflows been designed so that the maximum flows passed to treatment were those shown in the bottom row of the table, and had no discharge occurred until those flows were reached, the total volume discharged from the overflow at Brighouse would have been halved, and that at Bradford would have been reduced by nearly two-thirds.

Sampling and analysis

112. At Northampton, samples were taken from the stilling chamber just upstream of the measuring flume, the sampler normally coming into operation when the flow reached 1 m.g.d. (about 3 times the average dry-weather flow). In the first year, samples were taken at 5-min intervals for the first hour and thereafter at hourly intervals; subsequently the sampling interval was changed to 10 min for the first two hours and 20 min thereafter. In each case sampling ceased when

TABLE 9. Settings of overflows at Brighouse and Bradford

Criterion	Brighouse				Bradford			
	m.g.d.	g.h.d.	DWF	in/h	m.g.d.	g.h.d.	DWF	in/h
Foul sewer flow above which overflow operates	3.5	603	6.2	0.084	1.6	260	8.8	0.056
Setting of hypothetical overflow discharging same yearly volume as actual overflow	5.15	888	9.2	0.131	2.9	472	16	0.105
Estimated peak flow passed to treatment	6.6	1100	12	0.17	5	800	28	0.19

the flow fell below 1 m.g.d., and the timing cycle restarted when it rose above it again. The control circuit was automatically switched off when the 36 available bottles had been filled.

113. At Brighouse, samples were taken from the storm sewer. The sampler was designed to come into operation when the depth of flow was 2–3 in (equivalent to a flow of $\frac{1}{2}$–1 m.g.d.), but in practice there was generally a short, and sometimes a long, delay before the first sample was taken, the flow by then being appreciably greater than 1 m.g.d. At Bradford, samples of the overflowed storm sewage were taken from the stilling chamber upstream of the storm-sewer measuring flume, and the sampler came into operation when the depth in the chamber rose to 3–4 in (equivalent to a flow of 0.5–0.7 m.g.d.). At both of these sites the control equipment was designed for samples to be taken every 5 min for the first hour and at hourly intervals thereafter, but at both sites there were occasions when the 5-min programme continued beyond the first hour.

114. At Northampton, samples were normally collected by the Laboratory within 24 h of their being taken, and analysed within the next 24 h; they were stored at about 4°C for the intervening period. The longest delay would occur with a storm which started shortly after 9 a.m. on a Saturday morning—the samples would not then be collected before the following Monday morning, the analyses being carried out on the Tuesday. There was normally a shorter interval between sampling and analysis at the other two sites—at Brighouse samples were collected by the Borough Engineer's Department and the analyses were shared between the Brighouse Sewage Works and the Yorkshire Ouse River Board, while at Bradford the samples were delivered to the Public Analyst by staff from the City Engineer's Department.

115. The samples from each site were usually examined for 5-day biochemical oxygen demand at 20°C (BOD) and permanganate value (4 h at 27°C), and for contents of suspended solids and ammoniacal nitrogen. Many were examined for chloride content and loss on ignition of suspended solids. Because the design of the samplers was such as to reduce the risk of their becoming choked by large or stringy solids, gross solids tended to be excluded from the sample, and it appeared likely that this would lead to underestimation of the strength of storm sewage. A comparison was therefore made at Northampton between samples taken automatically and those taken by hand in such a way as to be representative of the whole sewage flow. (This exercise could only be safely carried out under dry-weather conditions.) It was found[3] that there were no significant differences between samples taken manually and automatically in the case of ammonia and chloride contents, but that, on average, the manual samples contained 15 per cent more suspended matter.

Composition of dry-weather sewage

116. Throughout the investigations at all three experimental sites, samples were taken of the sewage leaving the drainage area in dry weather. At Northampton 737 were taken by automatic sampler on 51 calendar days, but at neither of the other sites could the sampler be used because it was mounted over the storm sewer; samples were therefore taken manually—566 on 12 days at Brighouse, and 831 on 16 days at Bradford.

117. Dry-weather samples were generally taken at half-hourly intervals at Northampton and each was analysed; at the other two sites they were taken at 15-min intervals and hourly composite samples, bulked according to flow, were analysed. The sampling programme at each site covered all days of the week; any difference between weekdays and weekends was obscured by the inherent variability in the composition of sewage.

118. The average concentrations of the constituents examined are shown in Table 10 together with the average daily loads per person; both are derived from the total daily load, and the average concentration therefore takes account of the diurnal variations in flow. The data have not been adjusted to allow for differences in concentration between samples taken

TABLE 10. Estimated average composition of dry-weather sewage from Northampton (Nor), Brighouse (Brig), and Bradford (Brad)

Constituent	Concentration (mg/l)			Load (lb/person day)		
	Nor	Brig	Brad	Nor	Brig	Brad
Suspended solids	320	113	232	0.107	0.14	0.075
Permanganate value	78	52	86	0.026	0.057	0.028
BOD	311	199	257	0.104	0.28	0.084
Ammoniacal nitrogen	43	25	31	0.014	0.034	0.010
Chloride	91	—	60	0.030	—	0.020

manually and automatically. The concentration figures for suspended solids, BOD, and ammonia are greatest at Northampton and least at Brighouse; qualitatively these results are reasonable because of the high proportion of the dry-weather flow at Brighouse attributable to infiltration. The daily load per person is highest at Brighouse and lowest at Bradford (except for permanganate value). Again this is qualitatively reasonable, since the contributions from industrial effluents are believed to have been greatest at Brighouse and least at Bradford.

119. The generally accepted figure for BOD load per person is 0.12 lb/day. It has been shown[3] that, when allowance is made for the difference between manual and automatic sampling and for the effects of deposition in dry weather, the Northampton figure of 0.104 lb/person day becomes 0.124 lb/person day. It is estimated that the high value of 0.28 lb/person day at Brighouse results from a contribution from industrial sources equivalent to 0.14 lb/person day plus a domestic load of 0.14 lb/person day. The Bradford figure of 0.084 lb/person day is appreciably lower than expected, but might be attributable in part to the fact that many people residing in the drainage area (largely a housing estate) worked outside it; some deposition was observed in the Bradford system, but much less than at Northampton.

Composition of storm sewage

120. It is reasonable to suppose that the strength of storm sewage discharged from a sewerage system will depend, to a greater or smaller extent, on five principal factors.

1. *Strength of dry-weather sewage.* The storm sewage discharged from a system which, in dry weather, carries a strong crude sewage might be expected to be stronger than that from a system in which the dry-weather sewage is normally weak.
2. *Time of day.* For a given run-off the storm sewage should be weakest during the middle of the night, at which time the dry-weather flow is least and the crude sewage weakest.
3. *Flow.* The greater the flow the greater should be the dilution of crude sewage and the weaker the storm sewage to be expected.
4. *Time since start of storm.* The first flush of storm sewage may well be appreciably stronger than would be calculated from the effects of the previous factors; the initial discharge of rain water to the sewers probably contains the greatest proportion of material washed from impervious surfaces and the rising flow is more likely to scour any deposits from the sewers than is the subsequent falling flow; in addition, the initial rise may displace the normal flow of sewage in such a way as to increase temporarily the sewage load passing the sampling point.
5. *Time since previous storm.* The longer the time interval between storms the greater should be the accumulation of material likely to be washed into the sewers from the drainage area and the greater the accumulation of deposits in the sewers likely to be available for scouring by the rising flow.

121. By expressing storm-sewage loads as multiples of the dry-weather load, the assessment of which has already been discussed, the effect of variations in the strength of the dry-weather sewage between the different sites (Factor 1) can be largely eliminated. The effects of the remaining four factors are now examined in turn.

Variations with time of day

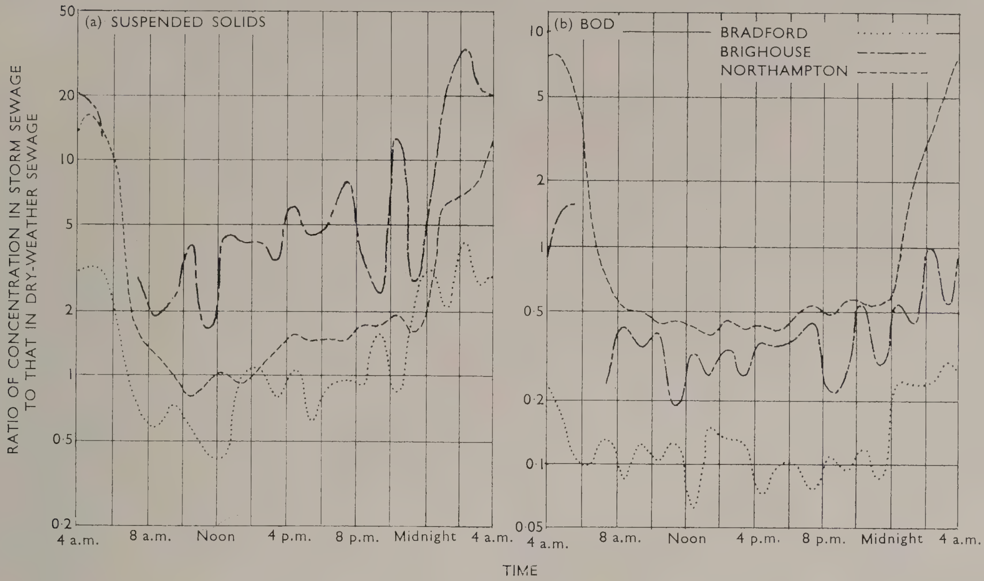
122. In general, it was found that storm sewage tended to be weakest at night, but the overall diurnal variations were far less marked than those of the dry-weather sewage[3].

123. To examine the relations between time of day and the strength of storm sewage in more detail, the average concentrations of suspended solids, permanganate value, BOD, and ammoniacal nitrogen in all the samples taken within each hour of the day were expressed as multiples of the corresponding hourly values found for the dry-weather sewage. The results for suspended solids and BOD are shown in Fig. 11. If storm sewage were crude sewage diluted by clean water, the average ratios over the 24 h should be appreciably less than the ratio of dry-weather flow to the total flow when sampling normally started—the latter ratio is roughly 0.3 for Northampton, and 0.1 for Brighouse and Bradford. The dry-weather flow being least during the night, the ratios should then be far below the values just quoted and correspondingly higher during the day. The departure of actual conditions from those suggested is evident from the figure. The storm/dry-weather ratio for suspended solids (Fig. 11(a)) varies between 0.8 and 16 at Northampton, between $1\frac{1}{2}$ and 30 at Brighouse, and between 0.4 and 4 at Bradford. The ratios for ammonia (not shown in the figure) varied between 0.04 and 0.5, values which indicate that the surface water contained an appreciable concentration of ammonia, but, on average, a lower concentration than that found in dry-weather sewage at any time of the day or night. Fig. 11(b) shows the ratios obtained for BOD, and a similar relationship was found for permanganate value; these parameters are affected by both suspended and dissolved matter, so it is reasonable that the ratios for them should lie between those for suspended solids and a soluble polluting constituent such as ammonia.

Variations with time since start of storm

124. For each site, all the data for each constituent measured during storms were grouped according to time since the start of the storm, and the average of each group has been plotted in Fig. 12. At Northampton the time was measured from the instant at which the flow first reached 1 m.g.d. (about 3 DWF) but successive storms were not considered to be separate unless the flow either fell below 1 m.g.d. for at least 4 h, or else fell to within 0.1 m.g.d. of the appropriate dry-weather value for at least 2 h. At Brighouse and Bradford a storm was regarded as starting at the

Fig. 11. VARIATIONS IN AVERAGE COMPOSITION OF STORM SEWAGE WITH TIME OF DAY.



instant the overflow came into operation, and storms were considered to be separate if discharge from the overflow ceased between them. Because of these differences in definition the results from the Northampton site are not directly comparable with those from the other two sites and are therefore shown separately on the left of Fig. 12.

125. Each point is plotted at the mid-point of the time range, except that the value for all samples taken more than 2 h after the start of a storm has been arbitrarily plotted at 150 min. Broadly speaking, the strength decreases with increasing time; this is particularly so at Northampton, for which the time origin is effectively further to the left than for the other sites. The results showing least change with time are those for the suspended-solids content at Brighouse, high concentrations being found even after more than 2 h. At each site the solids content during the first 5 min was significantly greater than in the dry-weather sewage; the averages for all samples taken after the first 35 min at Northampton and Bradford were three-quarters of the average dry-weather values, but at Brighouse the concentration remained at about four times the dry-weather value.

Variations with flow

126. Data for each constituent were also grouped according to flow. A detailed description of the method of averaging the data has been given in a published paper[3].

127. To provide a common basis for comparison of the results from the three sites, the difference between observed flow and dry-weather flow has been divided by the impermeable area to give values of i (defined by Equation 1) which are most conveniently expressed in inches per hour. The average results for each group of data, plotted at the mid-points of the flow ranges examined, are shown in Fig. 13 from which it is apparent that this method of analysis gives little information about the composition of storm sewage which could reasonably be considered to be generally applicable. The suspended-solids content is seen to increase slightly with increasing flow at Northampton and Bradford, and markedly at Brighouse. At values of i greater than 0.1 in/h the variations in BOD with flow are not great, and the results for permanganate value—not given in the figure—showed even less variation. The ammonia concentration falls with increasing flow at Northampton and Brighouse.

128. If the storm sewage consisted simply of crude sewage diluted with surface water, the concentration, C, of any particular constituent would be given by

$$C\,Q = Sq + W(Q-q) \qquad (7)$$

where S and W are the corresponding concentrations in the dry-weather sewage and the diluting water respectively, q is the dry-weather flow, and Q the total flow. In practice, both S and q are subject to large diurnal variations, and W may be expected to vary with the intensity and duration of rainfall and the length of the antecedent period of dry weather. Nevertheless, it is of interest to use averaged values for particular constituents and ranges of flow, and to see to what extent the experimental data support this type of relationship.

Fig. 12. VARIATIONS IN AVERAGE COMPOSITION OF STORM SEWAGE WITH TIME SINCE START OF STORM.

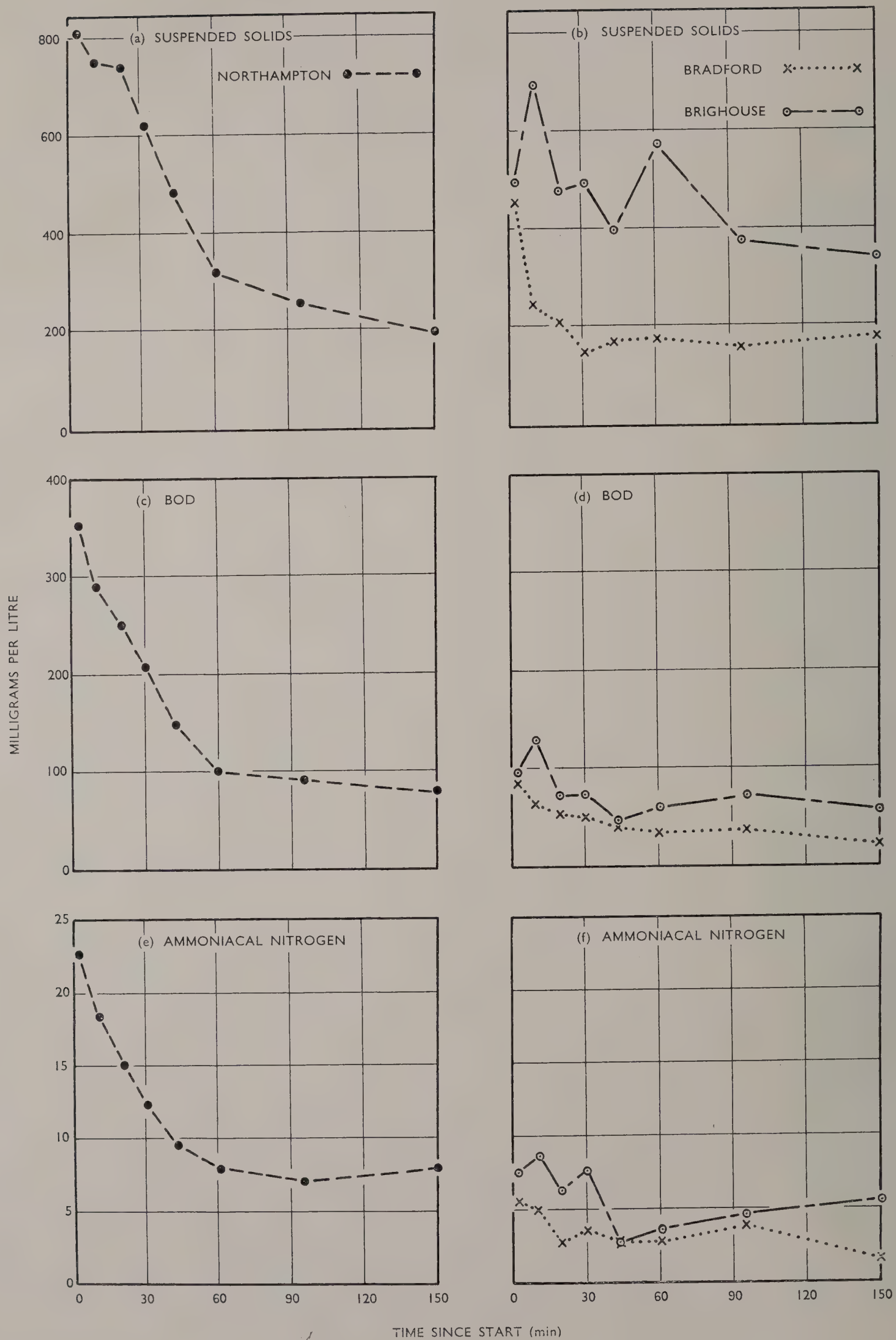
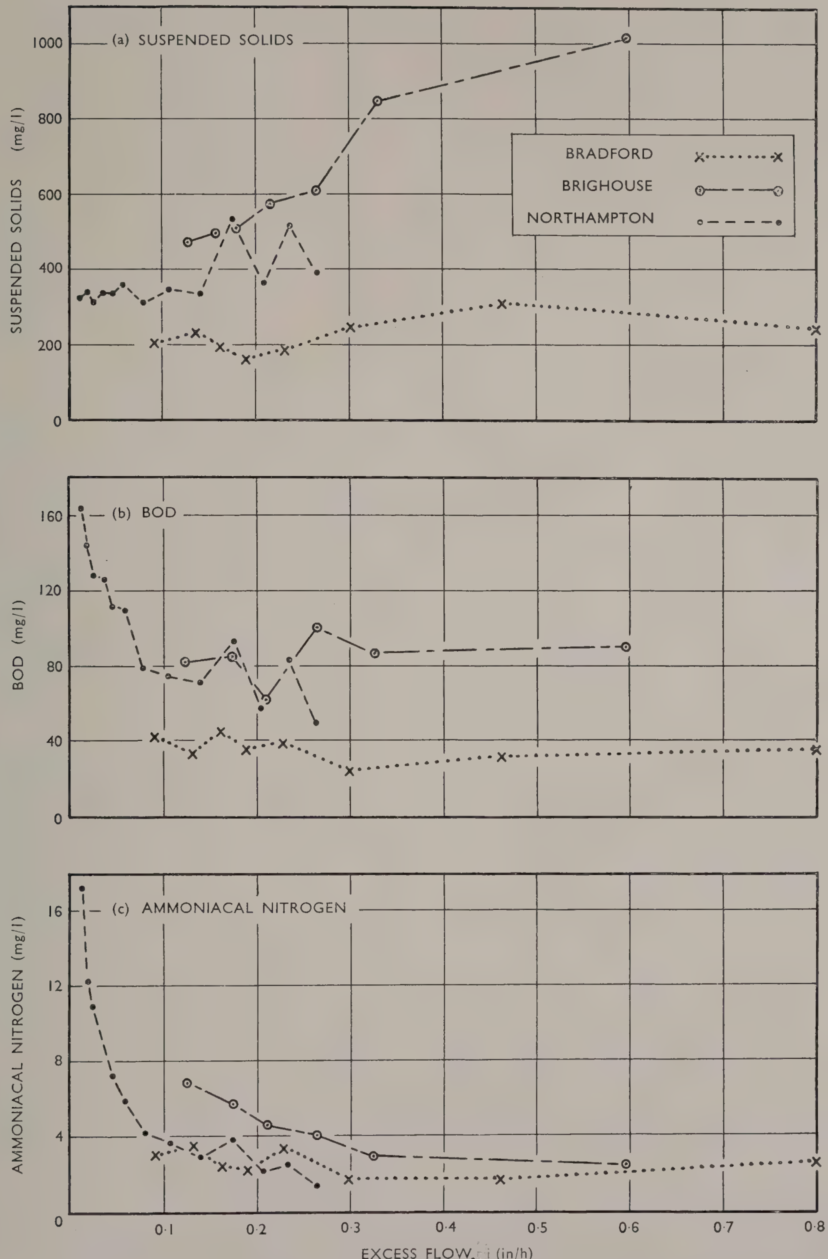


Fig. 13. RELATION BETWEEN AVERAGE CONCENTRATIONS OF CERTAIN CONSTITUENTS OF STORM SEWAGE AND EXCESS FLOW ATTRIBUTABLE TO RAINFALL PER ACRE OF IMPERMEABLE DRAINAGE AREA IN INCHES/HOUR.



129. Equation 7 may be re-written as

$$C=S-(S-W)\frac{Q-q}{Q} \tag{8}$$

Consequently, if C is plotted against $(Q-q)/Q$ a linear relation should be obtained; when no surface water is entering, Q has an average value of q and the equation reduces to $C=S$; when $(Q-q)/Q$ approaches unity, C approaches W. In Fig. 14(a) the results for the ammoniacal-nitrogen content of storm sewage at all three sites are plotted in this way. No results can be shown for low values of $(Q-q)/Q$ since, apart from those taken in dry weather, samples were only taken when the flow was over 3 DWF at Northampton or when the overflows were discharging at Brighouse and Bradford. The relations shown for Northampton and Brighouse are approximately linear; the Bradford data are more scattered. A linear equation has been fitted to each set of data by the method of least squares. The values of W, the concentration of ammonical nitrogen in the surface water, as found by putting $(Q-q)/Q$ equal to unity, are 1.3 mg/l for Northampton, 1.0 mg/l for Brighouse, and 1.9 mg/l for Bradford. Values of S, the concentration in dry-weather sewage estimated from the storm-sewage data by putting $(Q-q)/Q$ equal to zero, are 57 mg/l for Northampton (1.3 times the value determined by sampling dry-weather sewage), 56 mg/l for Brighouse (2.2 times the ascertained value), and 23 mg/l for Bradford (0.75 times). If an estimate were required of the average content of ammoniacal nitrogen likely to be discharged from a particular system at a particular flow, it would seem reasonable to use Equation 8, putting S equal to the ammoniacal-nitrogen content of the dry-weather sewage, and W equal to 1.4 mg/l (the average value found from these three sites).

130. It is apparent that the situation is very different when considering suspended matter (Fig. 14(b)) and that no single figure can be given for W since this varies from 200–300 mg/l at Bradford to over 1000 mg/l at Brighouse. Furthermore, Equations 7 and 8 are clearly not applicable to Brighouse because the data are not linear; a straight line fitted to the data gives a *negative* concentration of about 5000 mg/l for the crude sewage. At Northampton and Bradford the solids content of the storm sewage is virtually independent of flow, being on average substantially the same as that in the dry-weather sewage.

131. The forms of the distributions for BOD and permanganate value lie between those for ammonia and suspended solids. It is concluded that the concentration of ammonia is determined mainly by flow and by the composition of the dry-weather sewage and the surface water, but that the concentrations of the other constituents are greatly affected by local conditions.

Variation with time since previous storm

132. For each site the maximum values of suspended-solids concentration and BOD observed during each storm were plotted against the time which had elapsed since the previous storm. Such diagrams showed considerable scatter[3] but there was a general trend of increasing concentration with increasing time interval in most cases. Average values for three time intervals are shown in Table 11. Comparison of the figures given for suspended solids with the form of data plotted in Fig. 14(b) suggests that at Northampton the interval between successive storms is the dominant factor, whereas at Brighouse the magnitude of the flow is equally important; at Bradford the solids concentration appears to be little affected by either factor.

TABLE 11. Approximate average maximum values of suspended-solids content and BOD in samples taken in individual storms, for three intervals between successive storms

Time interval between successive storms	Suspended solids (mg/l)			BOD (mg/l)		
	Nor	Brig	Brad	Nor	Brig	Brad
1 hour	400	400	300	160	70	40
12 hours	700	700	260	280	120	50
5 days	1800	1000	330	600	180	60

Origin of suspended matter

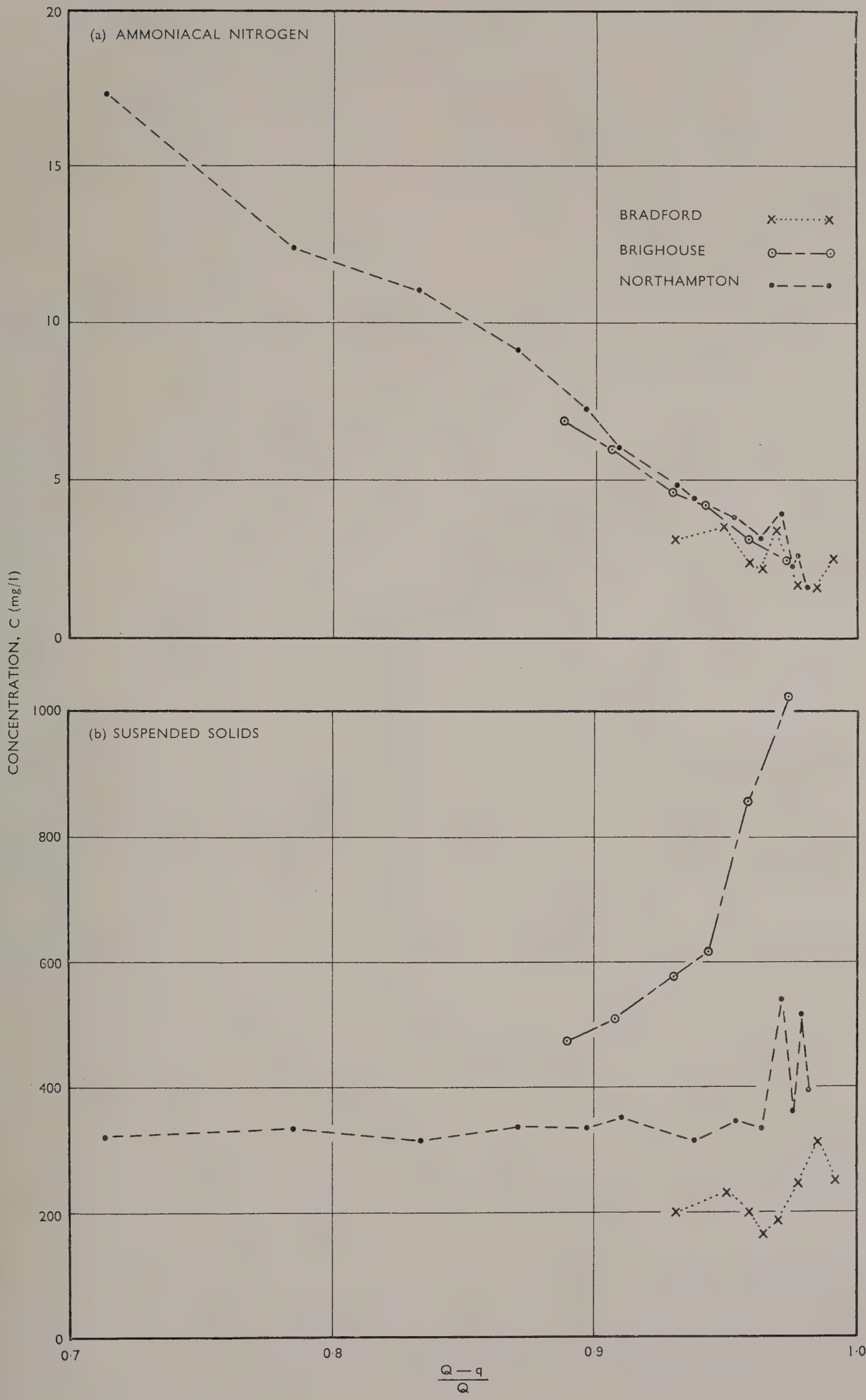
133. It will be apparent from the foregoing that although the concentration of ammoniacal nitrogen in storm sewage can be reasonably considered to result from the dilution of dry-weather sewage with storm run-off which itself contains a comparatively low concentration of ammoniacal nitrogen, the concentrations of suspended solids, and of other parameters the values of which depend to a certain extent on the solids content, cannot be so readily explained. Attempts were therefore made at each site to determine the origin of the solids present in the storm sewage.

Northampton

134. At Northampton—the first site to be examined in detail—concentrations of suspended solids in the storm sewage were much greater than had been expected. Representative sampling of surface water entering this combined system was considered impracticable, but a few samples were taken of surface water flowing from an adjacent area which was drained on the partially-separate system. Analyses of these suggested that although an appreciable weight of solid material probably entered in the first flush of surface water this alone could not have been responsible for the high concentrations found in the storm sewage.

135. The solids concentration in the surface-water samples taken at Northampton was lower than that in the dry-weather sewage and could not therefore have caused the concentration in a combined sewer to rise above the dry-weather value. Nevertheless on two occasions storms started when the automatic sampler had already been operating for several hours to obtain samples of dry-weather sewage, and on both of these the concentration of suspended solids rose by a factor of 3 to 4 while the flow was increasing.

Fig. 14. VARIATION IN AVERAGE COMPOSITION OF STORM SEWAGE WITH RATIO OF EXCESS FLOW TO TOTAL FLOW.



136. On a number of occasions there was a substantial discharge of clean water to the system from a reservoir in dry weather. For the thirteen occasions when such a discharge caused the flow to rise sufficiently for the sampler to operate, the concentration of suspended solids in the first two samples was, on average, a hundred times as great as would have been expected from dilution of the crude sewage, had this been the sole source of the solid matter. Towards the end of the investigation, clean water was flushed down about a mile of sewer in the early hours of the morning when the dry-weather flow was at its lowest; again, very high concentrations of suspended solids were found at the regular sampling point. Some months later, the velocity of flow in this mile of sewer was determined using a lithium salt as tracer; the average velocity during the night was found to be about 0.5 ft/s, and during the late morning 1.4 ft/s. It was also roughly calculated that the velocity of the night minimum flow was less than 0.5 ft/s for 38 per cent of the sections of sewer in the whole system, the corresponding figure for the peak daily flow being 15 per cent. It thus seemed likely that deposition would occur at many points during the night in dry weather, and at some points even throughout the day. Several sections of the sewerage system were examined for deposition, and samples of deposits at nine points were analysed; at these points the dry weight of the deposits averaged about $7\frac{1}{2}$ lb per ft run of sewer, and the volatile solids almost 1 lb/ft.

137. Taking all this evidence together, it seems reasonable to conclude that the initial rise in concentration of suspended matter at the beginning of storms at Northampton was largely caused by the scouring of deposits from the sewerage system, most of which consisted of old brick ovoid sewers of very generous dimensions. Numerous places were found where the mortar no longer filled the interstices between the bricks, and the resulting roughness combined with large sewer size probably assisted deposition. A more detailed account of this part of the investigations has been published[3].

Brighouse

138. The solids content of Brighouse storm sewage was exceptionally high; on average (see Table 13) it was nearly six times that of the crude sewage and was found to increase with the interval between successive periods of discharge (Table 11). The organic content was only about half that at Northampton, the ratio of suspended-solids concentration to BOD at Brighouse being roughly twice that at Northampton (Tables 11 and 13). The loss of weight of solids on ignition was determined for many of the Brighouse samples and for occasional samples from Northampton; the results for both dry-weather and storm sewage are compared in Table 12. The figures for crude sewage are seen to be identical but those for storm sewage at Northampton to be twice as great as at Brighouse. From an examination of the flow at which each sample was taken, it was concluded that only a small part of this difference could be attributed to the fact that sampling at Northampton started at 3 DWF compared with about 10 DWF at Brighouse. The Brighouse data for loss on ignition showed no correlation with time since start of discharge, but appeared to be related to flow—the average value for discharges of up to 2 m.g.d. from the overflow was 33 per cent, and for discharges greater than 5 m.g.d. was 28 per cent.

Table 12. Means and standard deviations of percentage loss of suspended solids on ignition. Figures in parentheses show number of analyses

	Northampton	Brighouse
Dry-weather sewage	(22) 78 ±9	(109) 78 ±10
Storm sewage	(35) 61 ±11	(287) 31 ±10

139. Little evidence of deposition of solids in the sewers was noted during the investigation, but it was over $3\frac{1}{2}$ years after the last sample of storm sewage had been analysed before a systematic examination of 30 of the 220 manholes in the sewerage system was carried out, those chosen being where conditions appeared to be most favourable for deposition. There were deposits in about half the manholes examined, but these did not normally extend more than a few feet from the manhole itself and were found, on analysis, to be largely inorganic.

140. It is concluded that, although some organic solids settle out in the Brighouse sewers, a high proportion of the solids borne by the storm sewage is inorganic and is probably carried into the sewers with the surface water during storms.

Bradford

141. At Bradford there were no large head sewers where deposition might be expected to occur, and little deposition was found. The results given in Table 11 suggest that if there was much accumulation of solid matter on the drainage area or in the sewers during dry weather then much of this material was carried past the overflow before the flow had risen sufficiently for the overflow to operate. However, Fig. 12 shows that the average suspended-solids content of the storm sewage during the first 5 min of discharge from the overflow was about twice that of the crude sewage; this indicates the entry of substantial quantities of solid matter in the surface water, or the scouring of deposits from within the system, and it is difficult to reconcile this with the lack of relation between the peak concentration found in individual periods of discharge and the interval between successive periods—it seems unlikely that the accumulation of solids in 1 h could not be very different from that in 5 days. Scouring of solids may well occur at much lower flows than that at which the overflow operates, in which case the time intervals that have been used are not the intervals between periods of scour (although there should be a correlation between them) and this, together with the inherent variability of the data, may thus mask any true effect of the interval between periods of scour and the peak solids concentration.

TABLE 13. Average composition of storm sewage discharged from a hypothetical overflow at Northampton set at 3.1 or 8.7 DWF, and of storm sewage discharged from existing overflows at Brighouse and Bradford. Concentrations in mg/l; figures in parentheses are percentages of dry-weather values.

Constituent	Northampton 3.1 DWF		Northampton 8.7 DWF		Brighouse		Bradford	
Suspended solids	368	(115)	391	(122)	637	(564)	237	(102)
Permanganate value	43	(55)	41	(53)	53	(102)	29	(23)
BOD	95	(30)	81	(26)	86	(43)	43	(17)
Ammoniacal nitrogen	5.5	(13)	3.8	(9)	4.9	(20)	3.1	(10)

Average composition of storm sewage

142. In Table 13 is shown the average composition of the storm sewage, weighted in terms of water volume, discharged from the overflows at Brighouse and Bradford, and that which would have been discharged from an overflow at Northampton set at 3.1 or 8.7 DWF (two of the settings for which detailed calculations were made), assuming that the flow to treatment remains constant while the overflow is discharging. The figures in parentheses in the table show the concentrations as percentages of the dry-weather value; with one slight exception the data for the two settings at Northampton, when expressed in this way, lie between the values for Brighouse and Bradford.

143. Although the work has shown that there are wide variations in the composition of storm sewage from site to site and time to time, it is perhaps worthwhile to suggest representative figures for its composition at the three sites. Straight averages of the four sets of data in Table 13 (expressed to one significant figure) would probably be the most appropriate—suspended solids 400 mg/l, permanganate value 40 mg/l, BOD 80 mg/l and ammoniacal nitrogen 4 mg/l. Averaging the figures in parentheses in the table gives corresponding values of 200, 60, 30, and 10 per cent of those of the average dry-weather sewage.

Polluting load

144. The total loads which would have been discharged as storm sewage in a year of average rainfall from a hypothetical overflow on the Northampton system at various settings between 3 and 30 DWF are shown in Table 14(a). The polluting loads estimated to be discharged from the Brighouse and Bradford overflows in a year of average rainfall (assumed in both cases to be 32 in, compared with 24.65 in at Northampton) are shown in Table 14(b) and (c) respectively.

145. It can be seen that the data of Table 14(a) are roughly applicable to the Brighouse and Bradford overflows when the settings of these overflows are expressed as the settings of the equivalent hypothetical overflows (which discharge the same yearly volume of storm sewage but restrict the flow to treatment to that at which discharge starts) divided by the average dry-weather flow excluding that of infiltration water. The ratios of the values given by Table 14(b) and (c) to the appropriate values found by interpolating in (a) vary from 0.4 (for permanganate value at Bradford) to 1.3 (ammonia at Bradford)—results which are probably as consistent as may be expected in view of the many uncertainties involved and the very different relations between flow and composition as illustrated by Fig. 13.

TABLE 14. Multiples of daily dry-weather load discharged from hypothetical overflow at Northampton, and existing overflows at Bradford and Brighouse, in year of average rainfall. Figures in parentheses calculated from Equation 6 and average composition of storm sewage

	Rainfall (in/year)	Overflow Setting: Nominal (multiples of total DWF)	Overflow Setting: Effective* (multiples of total DWF)	Overflow Setting: Effective* (multiples of DWF excluding infiltration)	Load discharged in year of average rainfall (multiples of daily dry-weather load): Suspended solids		Permanganate value		BOD		Ammoniacal nitrogen	
(a) Northampton	24.65	3	3	3	132	(228)	62	(69)	35	(34)	14.4	(11.4)
		6	6	6	91	(166)	42	(50)	21	(25)	7.8	(8.3)
		9	9	9	65	(112)	28	(34)	14	(16)	4.5	(5.6)
		12	12	12	51	(82)	21	(25)	9.4	(12)	3.0	(4.1)
		15	15	15	38	(61)	16	(18)	7.1	(9)	2.2	(3.1)
		20	20	20	26	(39)	10	(12)	4.6	(5.8)	1.3	(1.9)
		30	30	30	10.5	(18)	3.8	(5.3)	1.7	(2.7)	0.4	(0.9)
(b) Brighouse	32	6.2	9.2	19	27	(32)	6.0	(9.7)	4.5	(4.9)	2.6	(1.6)
(c) Bradford	32	8.8	16	16	32	(18)	5.7	(5.4)	2.4	(2.7)	1.1	(0.91)

* Setting of equivalent hypothetical overflow (Table 9).

146. A possible application of the results of this work is the use of Equation 6 coupled with the average composition of storm sewage (expressed as a percentage of the dry-weather composition), quoted in para. 143, to predict the annual discharge of polluting load from an overflow at any given setting. The figures in parentheses in Table 14 show, for comparison, the polluting loads calculated in this way. It is realized that both sets of figures were obtained from the same basic data, and that overall there should therefore be reasonable agreement. However, the comparison should give some idea of the deviations to be expected from the use of average values for the composition of storm sewage. As might be expected, the figures for the loads of suspended solids show the greatest discrepancies, but it is of interest to note that the agreement of the BOD loads is very fair throughout the whole range; this parameter is the one normally used in assessing the quality of river water or the effect of an effluent on the general health of a river.

147. To put in perspective the polluting loads discharged from an overflow, they may be compared with those which would be discharged from a sewage works. For the purpose of this discussion it will be assumed that all the sewage passing down the foul sewer is eventually treated at the works, from which it is discharged with a suspended-solids content of 30 mg/l and a BOD of 20 mg/l.

148. It is of interest first to calculate the addition to the dry-weather load discharged to the watercourse that would arise if all run-off were passed to the works and treated to this standard. Clearly, with any given standard of treatment (applying in both dry and wet weather), the additional load is directly proportional to the additional flow, which at Northampton is 42 per cent, at Brighouse 17 per cent and at Bradford 38 per cent of the dry-weather flow. (The low figure at Brighouse, when expressed in these terms, arises mainly because of the high dry-weather flow per head at that site.) The percentages discharged from the sewage works and the overflow for a number of overflow settings have been calculated from the Northampton data and are shown in Table 15(a). It may be noted that the values in the first column go beyond the 142 per cent which would be expected from the 42 per cent additional flow already quoted; however, this figure was derived from two years' data for that site, whereas the values given in the table were obtained from only the second year's and there were slight differences in the methods of calculation and of making allowance for incomplete data. In Tables 15(b) and (c) are shown the results calculated from the average figures for the suspended-solids content and BOD of storm sewage discharged from the existing overflows at Brighouse and Bradford.

149. The figures in the last two columns of Table 15 show the relative proportions of the total loads discharged from the overflows at Bradford and Brighouse to be very much less than for a hypothetical overflow at Northampton with the same settings. This is partly explained by the fact that the overflows at Brighouse and Bradford were not efficiently controlled and the flows to treatment were greater than those represented by the nominal settings of 6.2 and 8.8 DWF respectively, the spilled flows being correspondingly decreased. Further, the crude sewage was weaker at Brighouse and Bradford than at Northampton, and the calculations assume the same effluent composition rather than the same percentage removal of BOD and suspended solids at the works. Thus 30 mg/l suspended solids at Northampton is only 9 per cent of the concentration in the crude sewage, whereas at Brighouse and Bradford the corresponding proportions are 27 and 13 per cent respectively.

TABLE 15. Polluting loads of suspended solids and BOD discharged as storm sewage, in year of average rainfall, from hypothetical overflow at Northampton and existing overflows at Brighouse and Bradford. Results expressed in terms of average loads discharged from sewage works, assuming all flow passed to treatment is discharged with 30 mg/l suspended solids (SS) and 20 mg/l BOD

Overflow setting (multiples of dry-weather flow)	Load (as per cent of dry-weather load of sewage effluent) discharged from			Storm-overflow load (per cent of total load)	
	Works	Storm overflow			
	SS or BOD	SS	BOD	SS	BOD
(a) Northampton					
3	117	429	156	78	57
6	127	301	97	70	43
9	133	217	63	62	32
12	137	163	44	54	24
15	140	126	32	47	19
20	143	85	20	37	12
30	146	34	8	19	5
(b) Brighouse					
6.2*	116	33	7	22	5
(c) Bradford†					
8.8‡(A)	167	56	15	25	8
8.8‡(B)	135	27	7	16	5

* Effective setting (on criterion of volume discharged) of 9.2 DWF, equivalent to 19 DWF when infiltration not included.

† (A) including and (B) excluding data for periods of prolonged run-off.

‡ Effective setting (on criterion of volume discharged) of 16 DWF.

The largest loads discharged

150. The comparison of average rates of pollution tends to minimize the effects of individual storms, and it is therefore of importance to consider separately the largest loads discharged from the three systems. In Table 16(a) are shown the loads that would have been discharged from a hypothetical overflow at Northampton during a storm on 12th June 1961, the results for each constituent (including volume) being expressed as multiples of the average daily load passing the sampling point in dry weather. This storm was the largest (in terms of the volume of run-off) examined at that site, but it has not been ascertained whether it

Table 16. Estimated volumes and polluting loads discharged, in largest storms for which complete data are available, from hypothetical overflow at Northampton and existing overflows at Brighouse and Bradford. All values expressed as multiples of average daily values passed to treatment in dry weather

Site	Overflow setting (DWF)	Duration of discharge (h)	Discharge (multiples of average daily dry-weather values) Volume	Suspended solids	BOD	Ammonia
(a) Northampton	3	9.5	5.8	4.0	0.93	0.26
	6	7.3	4.8	3.4	0.73	0.15
	12	5.5	3.1	2.2	0.39	0.054
	20	3.4	1.7	2.1	0.16	0.009
(b) Brighouse	6.2	6.2	0.77*	3.9	0.17	0.056
(c) Bradford	8.8	9.0	2.77	2.5	0.45	0.22

* 1.6 if infiltration excluded, 3.1 if trade discharges also excluded in calculating dry-weather value.

Table 17. Summary of results of calculations on effect of storage on discharge of storm sewage at Northampton and Brighouse for overflows set at 6.2* DWF

Site (and number of storms considered)	Northampton (68)		Brighouse (29)		
Tank	A	B	C	D	E
Capacity of tank					
in thousand gal	27.8	83.3	23.3	46.7	140
as hours' dry-weather flow	2	6	1	2	6
in gallons per head	2.9	8.7	4.0	8.1	24.3
in inches of rain on impermeable area	0.011	0.032	0.016	0.032	0.095
Percentage of occasions					
tank partly filled	41	69	66	69	90
tank already full	—	—	7	7	7
tank completely filled	59	31	27	24	3
tank remaining full	—	—	10	10	10
tank partly emptied	12	22	10	10	14
tank completely emptied	88	78	80	80	76
Average filling time (min)					
for storms partly filling tank	26	33	17	18	24
for storms completely filling tank	22	43	12	16	146
for all relevant storms	24	36	15	16	26
Weighted mean strength of stored liquor (mg/l)					
in terms of suspended solids	783	662	774	817	764
in terms of BOD	226	218	113	114	102
in terms of ammoniacal nitrogen	11.8	10.5	6.1	5.6	5.3
Weighted mean strength of storm sewage discharged to river (mg/l)					
in terms of suspended solids	392	337	747	685	449
in terms of BOD	92	78	96	87	84
in terms of ammoniacal nitrogen	3.8	3.1	4.8	4.9	6.8
Discharge to river, with storage, as percentage of that without storage in terms of total					
duration	61	41	59	47	12
volume	79	59	64	45	2
suspended-solids load	65	41	63	41	1½
BOD load	60	37	60	39	2
ammoniacal nitrogen load	65	41	58	42	3
Equivalent setting (multiples of dry-weather flow) of overflow, without storage, to give same discharge to river in terms of					
frequency	13	16½	10	10½	17½
duration	9	11	8	9	14½
volume	7½	9½	11*	13*	24*
suspended-solids load	8½	12	8½	11½	29
BOD load	8½	11	8½	13	25
ammoniacal-nitrogen load	8	10	7½	8½	18

* In terms of volume discharged the Brighouse overflow is equivalent to one set at 9.2 DWF if flow to treatment were restricted to the overflow setting.

gives the largest values for the three highest settings of the overflow—the storm discharging the greatest loads for a setting of 3 DWF may not necessarily be that which would discharge the greatest for a setting of 20 DWF. The total excess discharge attributable to rainfall on this occasion was equivalent to 6.6 days of dry-weather flow, and with a hypothetical overflow set at up to 11 DWF more than half this volume would have been discharged as storm sewage. The figures for suspended solids are seen to be comparable with those for volume, thus indicating that the solids concentrations in the storm sewage were comparable with those in the dry-weather sewage. The ammonia loads, on the other hand, are all much less than the daily dry-weather load—this is to be expected because the flow remained above the dry-weather value for less than half a day and the chief source of the ammonia in storm sewage is that present in the crude sewage. The BOD loads, in the same units, lie between those for suspended solids and ammonia. It may be noted that, on this occasion, raising the overflow setting from 6 to 12 DWF would have reduced the volume of storm sewage and the load of suspended solids discharged by about 35 per cent, the BOD load by about 45 per cent, and the ammonia load by about 65 per cent.

151. In Table 16(b) are shown data for the largest loads estimated to have been discharged from the Brighouse overflow; these are the highest values for each particular constituent in any storm. At this site the occasion when the largest volume was discharged was also that for the largest load of each constituent, but at the Bradford site each of the last four figures shown in Table 16(c) relates to a different occasion.

Calculated effects of storage

152. The effect of providing storage for the storm-sewage discharge at a hypothetical overflow at Northampton has been calculated from data for the 68 storms which occurred in the six months from February to July 1961. The cases considered were for an overflow set at 2 m.g.d. (6.2 DWF), the initial discharge being to a storage tank with a capacity equivalent to either 2 or 6 hours' dry-weather flow, and the liquor retained in the tank being pumped back into the foul sewer as soon as the total flow dropped below 2 m.g.d.; the foul-sewage flow was assumed to be restricted to this value at all times, and the pump to be capable of maintaining the flow at 2 m.g.d. until the tank was empty. Similar calculations were made for the existing overflow at Brighouse, using the data for 29 periods of discharge during March–July 1960. Storage capacities equivalent to 2 and 6 h were considered as at Northampton, but since about half the dry-weather flow at Brighouse appears to consist of infiltration water the calculations were also made for a tank with a capacity of 1 hour's dry-weather flow. The existing overflow at Brighouse differs from the hypothetical one at Northampton in that the flow to treatment is not restricted to that at which discharge starts. The average flow above which the overflow operates is 3.45 m.g.d.; it has been assumed that pumping back from the tank to the sewer would be done by means of a pump delivering 1 m.g.d. and that the pump operates only when the flow in the sewer falls below 2 m.g.d. This is a more practical system than the one considered for Northampton, but it results in the storage tanks sometimes remaining full between storms whereas there would always be some emptying of the Northampton tanks.

153. The results for the various sized tanks at the two sites are compared in Table 17. It may be noted that the average setting of the Brighouse overflow was 6.2 DWF which was also the setting used in the Northampton calculations. However, there were two major differences between these sites; the dry-weather flow per person at Brighouse was about three times that at Northampton, and the flow to treatment at Brighouse rose to about 12 DWF during the heaviest storm—that at Northampton was assumed never to exceed the flow at first spill. The equivalent settings for Brighouse (Tanks C-E) in the last section of the table were derived on the assumption that at higher settings the flow to treatment would increase with increasing discharge from the overflow in approximately the same manner as with the existing overflow.

154. To reduce the results to terms of a single equivalent setting for each tank, the six estimates given in the last section of Table 17 have been averaged. The hypothetical overflow at Northampton had a setting of 6.2 DWF. The excess flow to treatment was thus 5.2 DWF (175 g.h.d.). This excess can also be expressed (using Equation 1) as 0.027 in/h. Provision of a 2-hours' DWF capacity tank would have been equivalent to raising the setting to 9 DWF (excess flow 270 g.h.d., or 0.041 in/h), whilst a 6-hours' DWF capacity tank would have been equivalent to raising the setting to 12 DWF (excess flow 370 g.h.d., or 0.057 in/h).

155. The Brighouse overflow had a nominal setting of 6.2 DWF and the excess was thus 5.2 DWF (500 g.h.d. or 0.083 in/h). Provision of a 1-hours' DWF capacity tank would have been equivalent to raising the setting to 9 DWF (excess flow 780 g.h.d. or 0.127 in/h), a 2-hours' DWF tank to 11 DWF (excess flow 970 g.h.d. or 0.16 in/h) and a 6-hours' DWF tank to 28 DWF (excess flow 2600 g.h.d. or 0.43 in/h).

Summary

156. These studies carried out at three drainage areas have amply demonstrated the difficulties of such research. At each site complications arose which made the interpretation of recorded data more difficult than had been expected. However, reasonably consistent results from flow measurements at Northampton and Brighouse, and their good agreement with earlier data obtained from Luton, suggest that Equations 1 to 6 might be usefully employed to predict the annual duration and volume of discharge from hydraulically efficient overflows in other areas. Because of the anomalous conditions encountered at Bradford the results from that site cannot be considered either to support or to refute those from the other sites. It should be remembered that the run-off must depend

upon the intensity of rainfall, and there is as yet little information about the distribution of low-intensity rainfall for most of the British Isles. It is likely that the results would not be applicable to areas having a rainfall pattern which differed markedly from those at the areas studied. Again, the times of concentration at the three areas studied were very similar, ranging from 12 to 23 min (40 min if Luton were included) and it might not be wise to extend predictions to drainage areas having times of concentration much outside this range. However, for all practical settings, the duration of overflow in a year is dependent more upon falls of low intensity and long duration rather than on short, intense storms, and the time of concentration is therefore likely to have relatively little influence upon the duration of overflow or the volume spilled.

157. The low side-weir overflows studied at Brighouse and Bradford were neither efficient in safeguarding the sewage works from overloading nor in preventing unnecessary pollution of the river. It would seem that at each of these sites an overflow capable of restricting the flow to treatment to that of first spill could have been set considerably higher than those installed, without increasing the maximum flow already taken to treatment. Such an overflow could have reduced the volume spilled by half at Brighouse and by nearly two-thirds at Bradford. The corresponding reductions in polluting load spilled would probably be even larger, as more of the first flush could be expected to be retained in the sewer.

158. The results of studies of composition of storm sewage were far less consistent than those of flow. However, it was found that storm sewage was weakest during the night, although there was less diurnal variation than observed in the dry-weather sewage; its strength decreased with time during storms, and to some extent depended upon the flow. It was concluded that the concentration of ammonia in storm sewage was determined mainly by the flow and by the composition of the dry-weather sewage and the surface water, but that the concentrations of the other constituents examined were greatly affected by local conditions such as deposition within the sewer during dry weather. At Northampton and Brighouse the composition of storm sewage was influenced by the length of the antecedent dry period, but at Bradford this appeared to have little effect.

159. For comparison of results from different drainage areas, the load of storm sewage spilled in a year of average rainfall from a storm overflow can be expressed as a multiple of the dry-weather load. The overflow setting can be expressed as that of an equivalent hypothetical overflow (which discharges the same yearly volume of storm sewage but at which the flow to treatment is restricted to that at first spill) in terms of multiples of the dry-weather flow excluding infiltration. Provided that the overflow settings at Brighouse and Bradford were expressed in this way, the estimated loads discharged from these overflows agreed reasonably well with those which would have been discharged from overflows at comparable settings at Northampton.

160. Although there were wide variations in the composition of storm sewage from site to site and from time to time, a rough estimate of the average strength of storm sewage found in these areas is 400 mg/l suspended solids, 40 mg/l permanganate value, 80 mg/l BOD, and 4 mg/l ammoniacal nitrogen. These averages are equivalent respectively to 200, 60, 30 and 10 per cent of the corresponding values for crude sewage, and if they were to be used to estimate the strength of storm sewage at other sites it might be preferable to express them in this way.

161. It was calculated that provision of tanks with capacities of either 2 or 6 hours' dry-weather flow at Northampton would have been equivalent to raising the setting of a hypothetical overflow from 6.2 DWF to 9 or 12 DWF respectively in the absence of storage. At Brighouse, providing tanks of 1, 2 or 6 hours' capacity at the existing 6.2 DWF overflow would be equivalent to raising the setting to 9, 11, or 28 DWF.

References

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2. Gameson, A. L. H., Davidson, R. N., and Threlfall, J. M. Storm flows from combined sewerage systems in three areas. J. Instn. Publ. Hlth. Engrs., 1965, **64**, 182–198.
3. Gameson, A. L. H. and Davidson, R. N. Stormwater investigations at Northampton. J. Proc. Inst. Sew. Purif., 1963(2), 105.

CHAPTER 4. LABORATORY EXPERIMENTS OF MODELS OF STORM OVERFLOWS

162. An efficient storm overflow should start to spill when the flow to the treatment works reaches a predetermined rate, this rate should not be exceeded with increasing flow upstream of the overflow and the maximum quantity of polluting material in the sewage should be carried forward to the treatment works. The experiments described in this chapter were aimed at assessing the extent to which different types of overflow fall short of these requirements[1].

163. The overflows studied were a low side-weir, a stilling pond, a vortex with spill over a central ring weir, and an overflow with storage beyond high side-weirs. The experiments were carried out with the upstream pipe at gradients of 1 in 500 and 1 in 100, the velocity of flow in the former being sub-critical and in the latter, super-critical. (Normally, where velocities are sub-critical, flow conditions at a given point are determined by conditions downstream; where velocities are super-critical, flow conditions are independent of conditions downstream.)

164. The main purpose of the laboratory experiments was to compare the effectiveness of the four types of overflow with non-steady polluting discharges and to assess the extent to which storage of a "first flush" of heavily polluting storm sewage might be beneficial. The scale of the apparatus was rather small for predicting accurately the performance of full-size structures, but this was not considered a serious drawback because a separate investigation on a field scale was carried out as described in Chapter 5.

165. Additional experiments were carried out on a vortex overflow with peripheral spill. These were of limited scope and were undertaken essentially as a preliminary to the more detailed investigation that would be required to establish the usefulness of such a structure. Nothing which can usefully be included in this report emerged from these experiments.

Description of apparatus

166. The apparatus consisted of a 250-ft length of 3-in diameter pipe, which may be considered as a 1/12 scale model of a 3-ft diameter combined sewer. A saline base flow (representing dry-weather sewage) was introduced at a constant rate into the upstream end of the pipe, together with particles of bakelite, polystyrene and polythene, to simulate grit, coarse suspended solids and floating solids respectively. Fresh water, to simulate surface or storm water, was added separately at a rate varying with time.

167. The model overflows were installed in turn at the downstream end, and the "flow to treatment" and "overspilled storm sewage" were collected separately. The amounts of "sewage" in the "flow to treatment" and in the "overspilled storm sewage" were assessed by measuring their salinity.

Description of the model overflows

168. The models were intended to be typical examples of overflows currently in use; details are shown in Fig. 15 and the following description indicates briefly their main features.

Low side-weir General dimensions were obtained from a study of various existing structures. The length of the overflow was 10 times the diameter of the upstream pipe and the diameter of the outgoing pipe was half that of the incoming one.

Stilling pond The design was based on an existing structure where there is only limited head available between upstream and downstream sewers, the reduction in velocity being achieved by increase in width rather than depth. Rate of discharge was controlled by an adjustable orifice.

Vortex with central weir The design was based on preliminary information about a type of overflow developed at Bristol, but did not incorporate some of the refinements which have been shown to be advantageous in more recent experiments[2]. Rate of discharge was controlled by interchangeable orifices.

Storage-type overflow The overflow was similar to one recently designed for a local authority and was basically a long rectangular storage chamber with an orifice at the downstream end to control the flow to treatment, and high side-weirs at the upstream end. The principle of such an overflow is that the dry-weather sewage present in a system at the onset of a storm should by-pass the overflow into storage downstream before any spill occurs over the weir—generally described as "storing the first flush".

Flow details

169. The base flow or dry-weather flow (DWF) was fixed as 1/50 of the inlet pipe capacity when flowing full without surcharge, so that the conditions could be regarded as simulating a sewer of 50 DWF capacity. The overflow weir levels were fixed so that first spill would in theory occur when the flow to treatment was at the rate of 5 DWF and (with the exception of the low side-weir, which had no controlling orifice) maximum flow to treatment would in theory not exceed 6 DWF. The discharges considered in the design calculations were:

	1 in 500	*1 in 100*
Base flow or dry-weather flow (DWF)	0.001 cusec	0.0025 cusec
Discharge to treatment at first spill (5 DWF)	0.005 ,,	0.0125 ,,
Discharge to treatment (6 DWF) at peak incoming discharge	0.006 ,,	0.015 ,,
Peak incoming discharge (50 DWF)	0.050 ,,	0.125 ,,
Peak spill over weirs (44 DWF)	0.044 ,,	0.110 ,,

Fig. 15. DETAILS OF SMALL-SCALE MODEL OVERFLOWS.

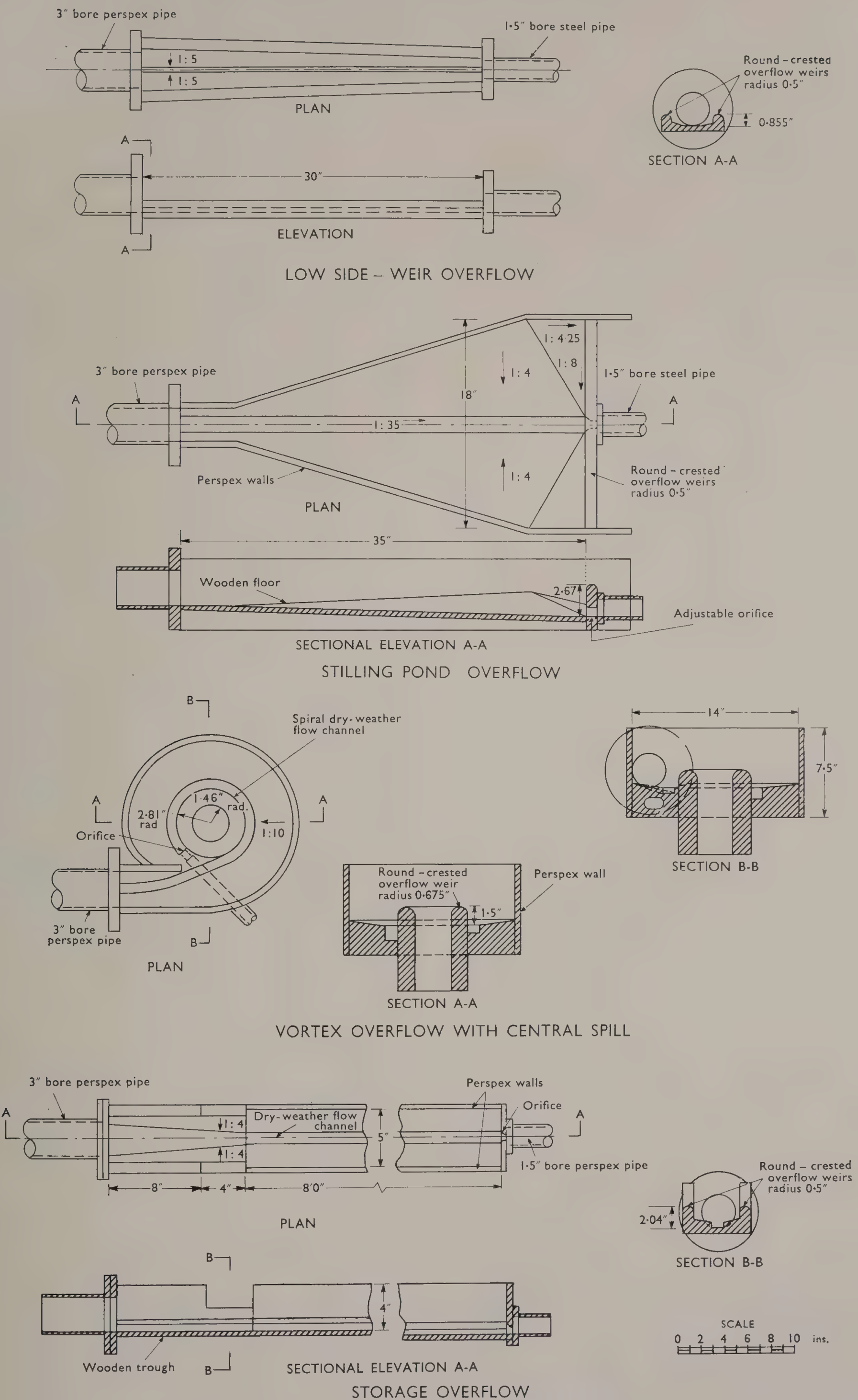
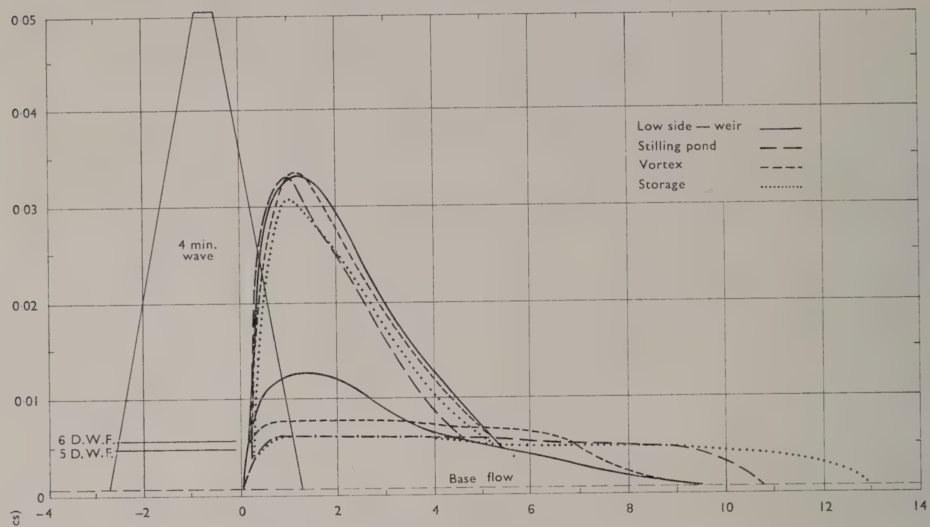
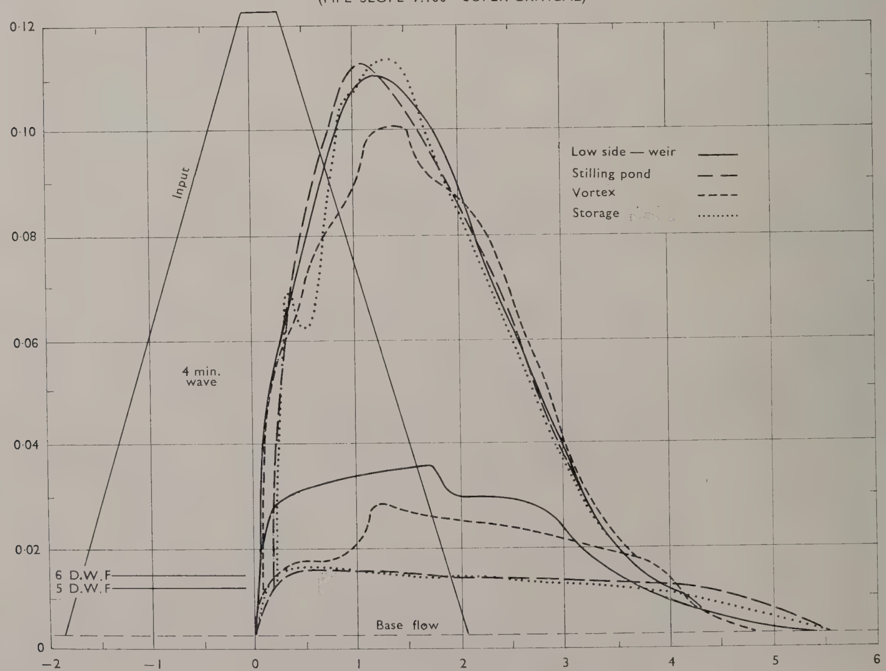


Fig. 16. DISCHARGE – TIME CURVES
SMALL-SCALE MODEL OVERFLOWS.

(PIPE SLOPE 1:500 SUB-CRITICAL)



(PIPE SLOPE 1:100 SUPER-CRITICAL)



170. The storm discharge was introduced to give waves of trapezoidal shape, the discharge increasing uniformly for 5/11 of the wave duration, remaining constant for 1/11 and then falling uniformly for a further 5/11, and four wave durations were considered, namely 1, 2, 3 and 4 minutes, corresponding to about $3\frac{1}{2}$, 7, $10\frac{1}{2}$ and 14 minutes in a 36-in diameter sewer. Overflow dimensions and levels were fixed to ensure that the upstream pipe was never surcharged.

Analysis of results

Hydraulic performance

171. Figure 16 is an example of the discharge-time curves obtained for each overflow structure, the upper lines showing the total discharge leaving the overflow and the lower lines the discharge to treatment. Notes on the hydraulic performance of the individual overflows are given in Table 18.

TABLE 18. Summary of hydraulic performance

Overflow structure	Gradient 1 in 500 (sub-critical)	Gradient 1 in 100 (super-critical)
Low side-weir	Only overflow to spill during the shortest flood. At high flows passed more flow to treatment than the others. Poor control. Flow to treatment reached 12 DWF.	For all flood durations, passed more flow to treatment than the others. Poor control. Flow to treatment reached 13 DWF.
Stilling pond	Good control. Maximum flow to treatment close to nominal 6 DWF as designed.	Good control. Maximum flow to treatment close to nominal 6 DWF as designed.
Vortex with central weir	Maximum flow to treatment about 8 DWF.	Erratic behaviour due to high entry velocities. Poor control. Flow to treatment reached 12 DWF.
Storage-type	Good control. Maximum flow to treatment close to nominal 6 DWF as designed. Spilled less of storm than the others (storage effect).	Good control. Maximum flow to treatment close to nominal 6 DWF as designed. Some reflection of flood wave and surging (see upper line Fig. 16).

Discharge of polluting material

172. Figures 17 and 18 show the concentrations of pollutants in the spill as proportions of the base flow concentration and significant points noted are indicated in Table 19.

Summary

173. It should be noted that the overflows were not, and were not intended to be the best of their respective types so far as detailed design and dimensions are concerned; they could probably have been modified to give better performance—for example, by enlarging them to improve separation characteristics. On the other hand, some actual storm overflows of the types tested may perform less efficiently.

TABLE 19. Summary of performance with polluting material

Overflow structure	Gradient 1 in 500 (sub-critical)	Gradient 1 in 100 (super-critical)
Low side-weir	Worst of the four with grit and coarse solids. Good retention of floating solids with scum-boards.	Moderate performance with dissolved pollution, coarse solids and grit, partly as result of poor control. Best retention of floating solids with scum-boards.
Stilling pond	Best of the four with coarse solids and grit. Good retention of floating solids with scum-board.	Moderate performance with all pollutants. No outstanding features. Small improvement in retention of floating solids with scum-board.
Vortex with central weir	Poor retention of floating solids. Very little improvement with scum-board. Otherwise moderate.	Worst of the four with dissolved pollution. Otherwise as for stilling pond.
Storage-type	Best of the four with dissolved pollution. Best retention of floating solids with and without scum-boards. Otherwise moderate.	Moderately good with dissolved pollution, especially at short storm durations. Best of the four with coarse solids. Best retention of floating solids without scum-boards, but no improvement with scum-boards.

174. The low side-weir was inefficient as a device for hydraulic separation and control. Control of flow to treatment under conditions of high flow was poor. Under sub-critical conditions there was a tendency for the bottom layers of flow to spill and thus excessive amounts of grit and suspended solids passed over the weir. In this respect, however, there was an improvement in performance under super-critical conditions. Furthermore, under these conditions, when fitted with scum-boards, it was the most efficient in passing floating solids to treatment.

175. The degree of hydraulic control exerted by the stilling pond was good and met the design requirements under all conditions. Under sub-critical conditions, when fitted with a scum-board, it gave the best overall performance in relation to retention of all solids. Under super-critical conditions, however, its efficiency in separating solids deteriorated considerably. (This was due to the greater loading, coupled with the turbulence created by a hydraulic jump near the entrance to the chamber.)

176. The degree of hydraulic control exerted by the vortex with central weir was not good, and under super-critical conditions its hydraulic performance was almost as bad as that of the low side-weir. It is possible, however, that more accurate control could have been achieved if full account of the effect of circulation had been taken in design. Its performance in retaining polluting material was only moderate.

Fig. 17. AVERAGE CONCENTRATIONS OF POLLUTANTS IN SPILL AS PROPORTIONS OF BASE-FLOW CONCENTRATION.

(PIPE SLOPE 1:500)

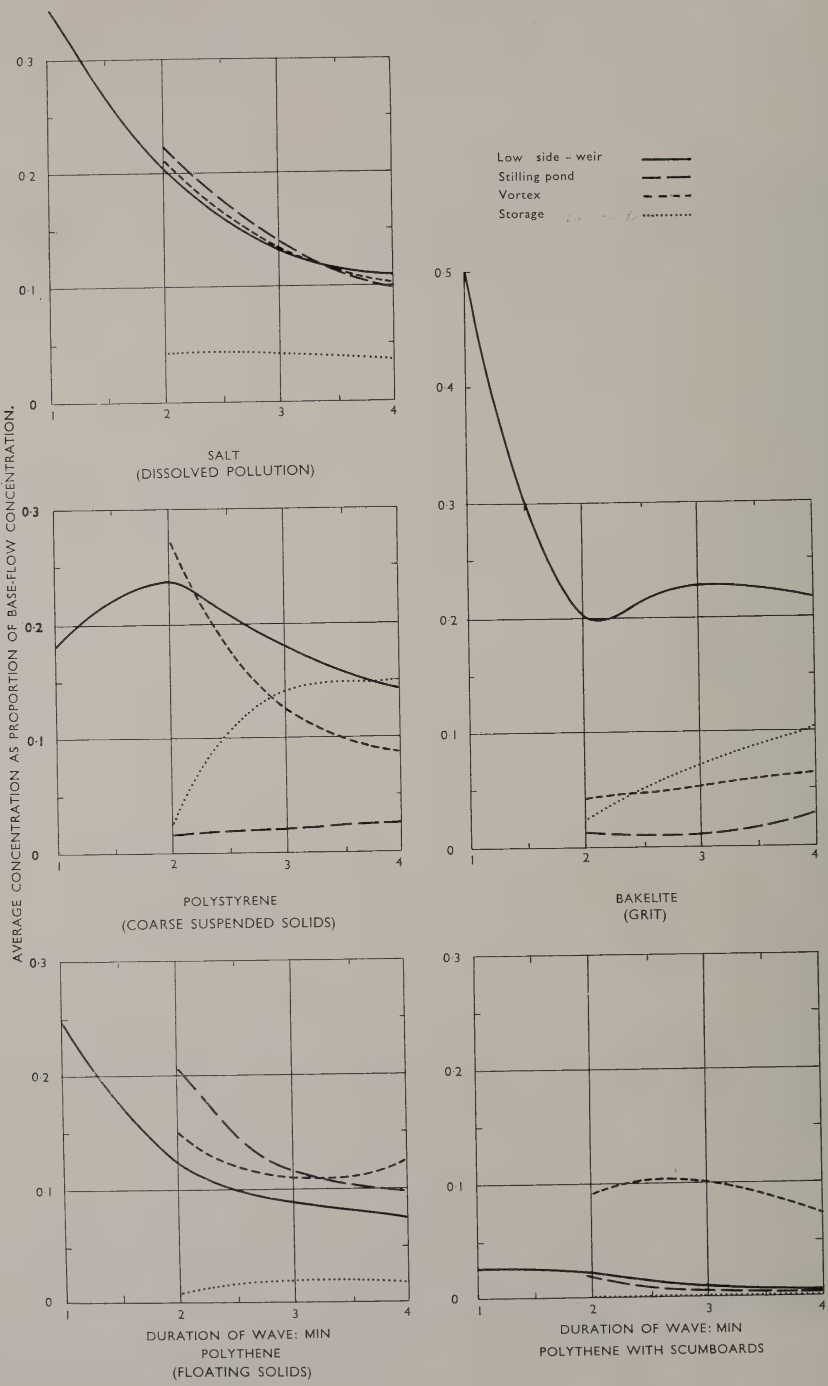
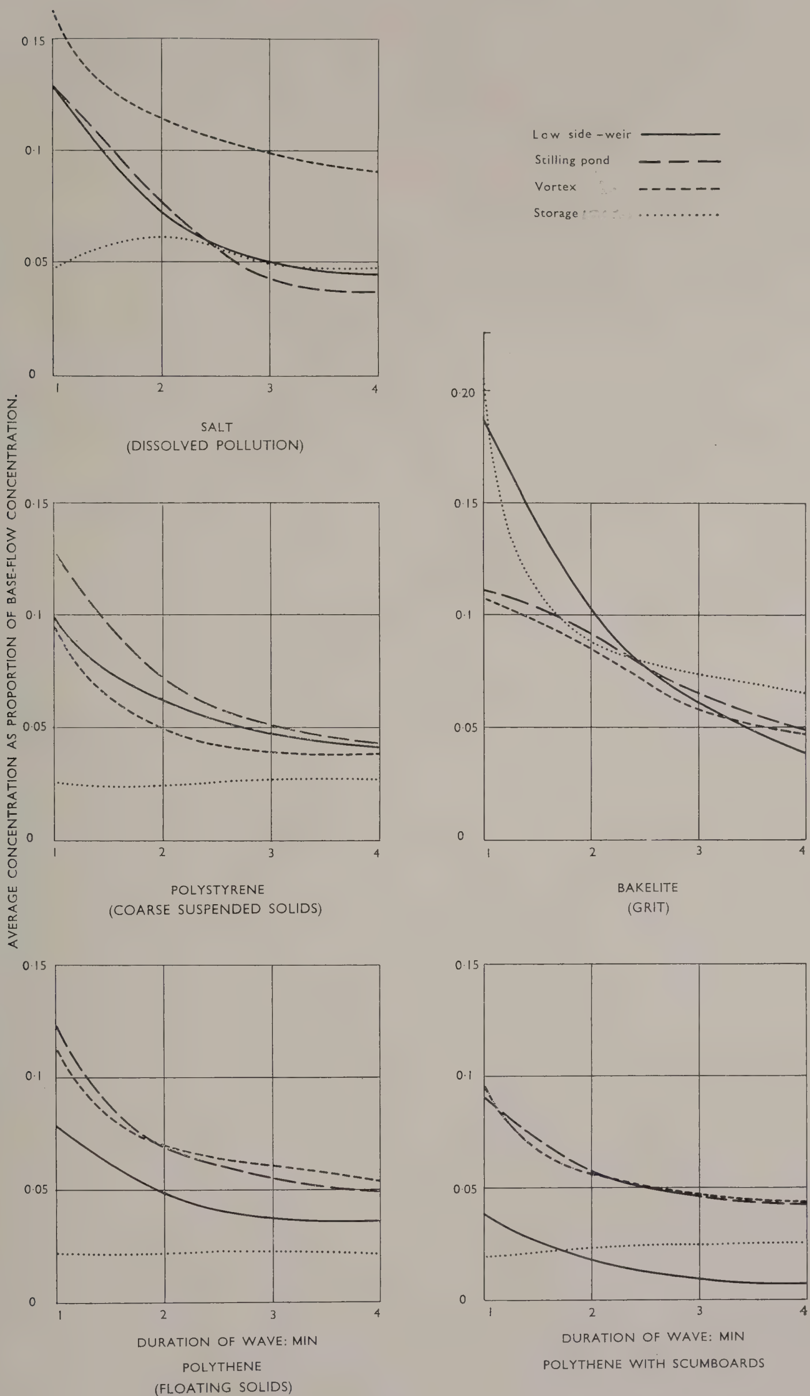


Fig. 18. AVERAGE CONCENTRATIONS OF POLLUTANTS IN SPILL AS PROPORTIONS OF BASE-FLOW CONCENTRATION.

(PIPE SLOPE 1:100)



177. The degree of hydraulic control exerted by the storage-type overflow was good and met the design requirements. Under sub-critical conditions it performed well in containing the "first flush" and so less of the dissolved (or finely suspended) polluting material passed over the weir. Under super-critical conditions, however, its advantage of storing the first flush was much less noticeable. (This was partly due to turbulence caused by reflection of the storm wave and back flow in the chamber and to the fact that the volume of the storm wave was greater in relation to the first flush.)

178. The experiments emphasised the importance of a sub-critical sewer gradient upstream of a storm overflow. With the exception of the low side-weir, all the overflows gave a worse performance under super-critical conditions and many of the advantages of the stilling pond and storage-type overflows were lost or reduced.

179. Although all types tested might be improved to give better individual performance, it seemed that the best storm overflow would be a combination of a storage-type, with the chamber at the head designed as a stilling pond with a scum-board. The approach sewer should be laid at a sub-critical gradient.

180. Chapter 5 deals with field-scale experiments on storm overflows and includes at the end a brief comparison between the laboratory-scale and the field-scale tests (paras. 201–203).

References

1. Ackers, P., Harrison, A. J. M., and Brewer, A. J. Laboratory studies of storm overflows with unsteady flow. Symposium on storm sewage overflows. Institution of Civil Engineers, 1967 (p. 37).
2. Smisson, B. Design, construction and performance of vortex overflows. Symposium on storm sewage overflows. Institution of Civil Engineers, 1967 (p. 99).

181. Because of the limitations of model testing and the practical difficulties associated with testing the performance of storm overflows in service, we looked for a site where it might be possible to investigate the properties of different types of overflow structure using crude sewage. We found a suitable site at the East Hyde Sewage Treatment Works of Luton Corporation and this chapter describes experiments carried out there[1]. In these experiments, we were particularly interested in hydraulic performance and the separation of gross solids.

182. The small-scale models described in Chapter 4 were tested under flood-wave hydrograph conditions simulating the superimposition of storm-water run-off on normal sewage flow, but it was not possible to create such conditions on the Luton site. The structures, comprising a low side-weir, a stilling pond, a vortex with spill over a central ring weir, and a high side-weir, were tested under steady flow at various rates. Because of these limitations, it was not appropriate to study a storage overflow.

Experimental site and method of operation

183. The general arrangement of the experimental site is shown in Plates 1 and 2. The main sewage carrier at the inlet of the treatment works is an elevated rectangular open concrete channel (Plate 1) passing close to the storm tanks. Crude sewage was drawn from this carrier and passed through the overflow structures built inside one of the storm tanks (Plate 2).

184. The tests were designed for flows of 1, 2, 3, 4 and 5 m.g.d., the sewage flow being drawn from the carrier through five siphons of flexible plastic pipe, each with a capacity of approximately 1 m.g.d. They discharged to a steel tank from which the sewage passed to the storm tank through a length of about 250 feet of 18-in diameter concrete pipe laid at the gradient to give a pipe-full capacity of 5 m.g.d.

185. Inside the storm tank, a sectional steel tank was erected on low walls and the overflow structures to be tested were supported in turn above the tank. The outgoing flows to treatment and to spill passed separately through screens of 3/16-in rectangular bars with ¼-in apertures, Sutro weirs being used for velocity control and flow measurement. Swinging troughs were used to direct the flows, as required, to tanks from which samples could be taken for subsequent analysis.

Description of overflows

186. The overflows are shown in Fig. 19 and it will be noted that the low side-weir, stilling pond and vortex structures were similar to the small-scale models described in Chapter 4.

187. The overflow weirs were fixed so that first spill would in theory occur when the flow to treatment was at a rate of 0.5 m.g.d. This was conceived as being 5 DWF for the purpose of the experiments, to enable the results to be expressed in terms which are generally understood. For all but the low side-weir, the design was such that maximum flow to treatment would not in theory exceed 0.6 m.g.d. (6 DWF). In the case of the low side-weir, the usual assumption was made that the level of the weir should coincide with the normal water level in the downstream sewer when carrying the first-spill flow. In the stilling pond and vortex, control of flow to treatment was by a rectangular orifice. The high side-weir was a modification of the low side-weir, the weirs being raised and shortened and a rectangular orifice installed to provide positive control of flow to treatment. Scum-boards were designed for each structure to be placed at a distance from the weir equal to the maximum head over it, and extending one-tenth of the depth of flow below the weir. In the case of the stilling pond, the scum-board could be placed in two alternative positions—6 in and 18 in from the weir.

Analysis of results

Hydraulic performance

188. The discharge characteristics of the overflows are shown by Table 20 and Figs. 20(a) and 20(b).

TABLE 20. Flows at first spill and at maximum inflow studied (All flows expressed in multiples of DWF)

Type of overflow	Flow at first spill	Flow to treatment at maximum inflow	Maximum* inflow
Low side-weir	2.8	7.8	36
Stilling pond	4.9	5.8	41
Vortex	5.4	6.8	38
High side-weir	5.2	6.2	39

* Practical difficulties prevented the structures from being tested at the maximum design flow of 5 m.g.d. (50 DWF).

The following observations were made:

Low side-weir The weirs were used inefficiently, most of the spill occurring over the last 3 ft of weir with hardly any spill over the upstream end. This was due to the fact that flow in the trough between the weirs was super-critical with a hydraulic jump at the downstream end. Scum-boards had no significant effect on the accuracy of control. Occasionally there was a decrease of flow to treatment caused by accumulations of rags partly blocking the mouth of the 9-in downstream pipe.

Stilling pond The discharge to treatment at first spill and the increased flow to treatment at higher inflow rates were close to the design figures when the scum-board was omitted. The

Plate I. MAIN SEWAGE CARRIER AND SIPHON DRAW-OFF PIPES.

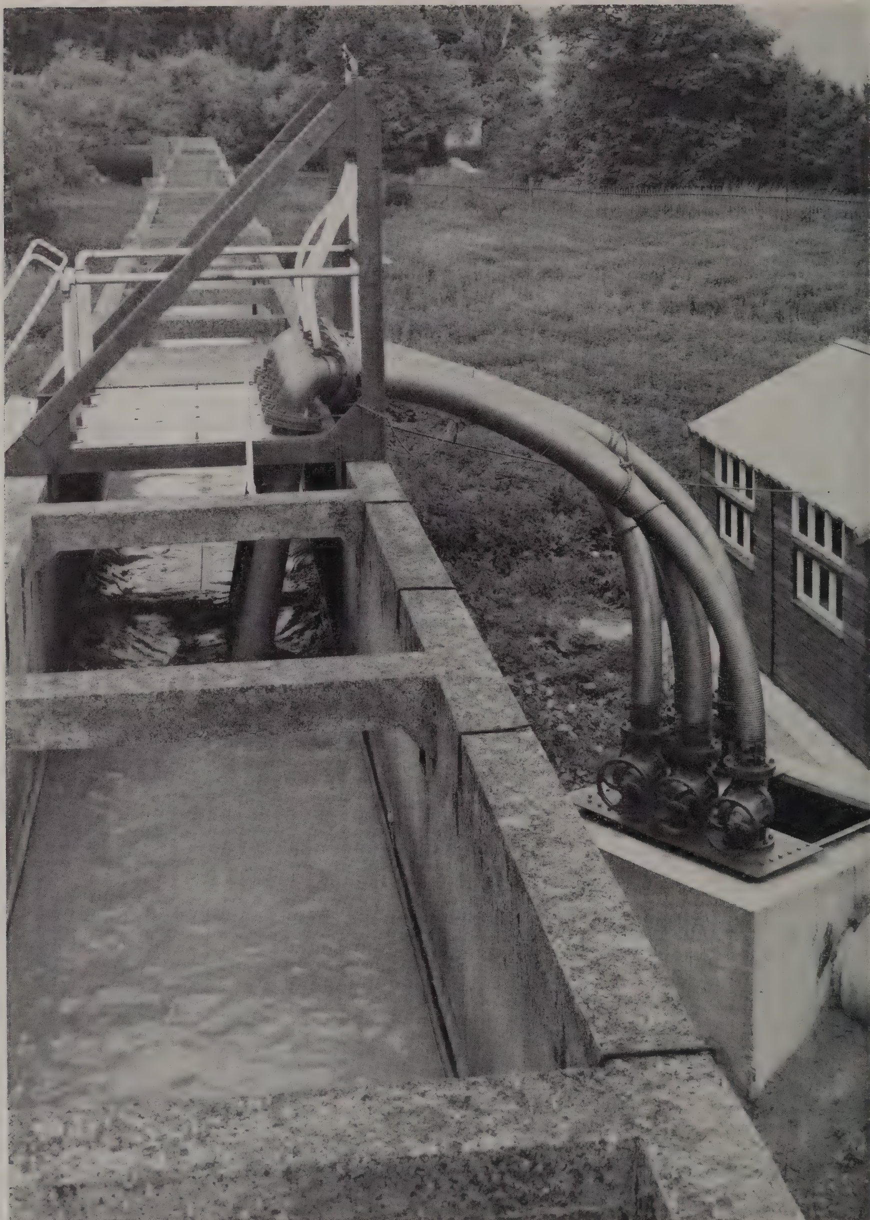


Plate 2. GENERAL ARRANGEMENT INSIDE STORM TANK SHOWING HIGH SIDE-WEIR OVERFLOW UNDER TEST.

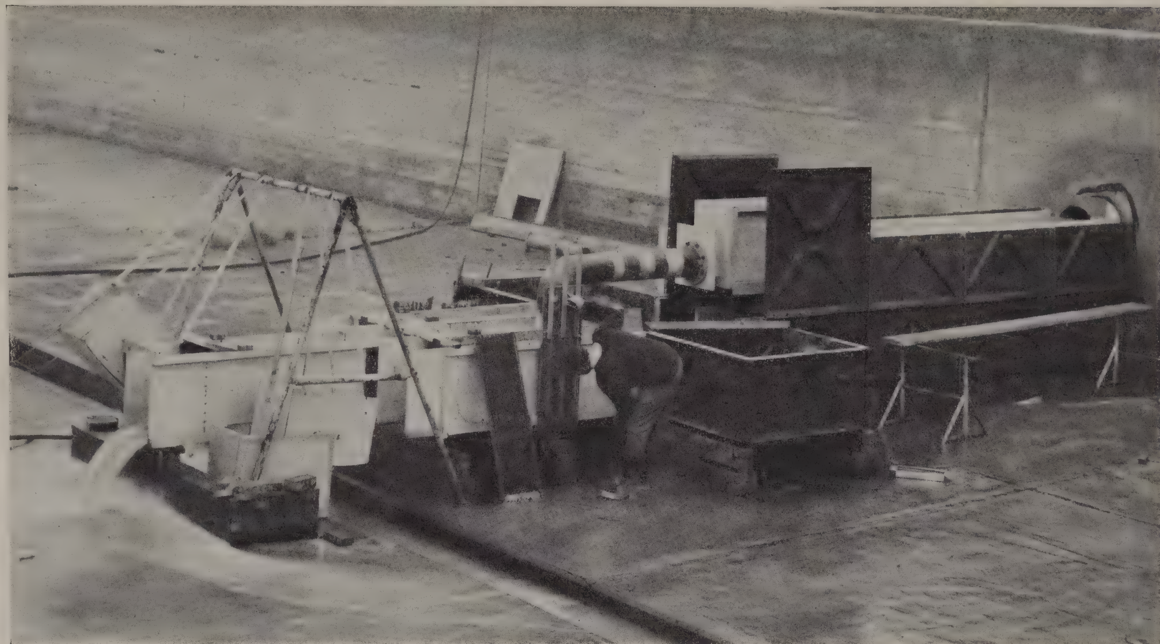
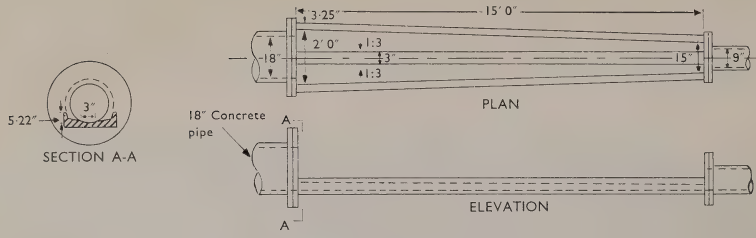
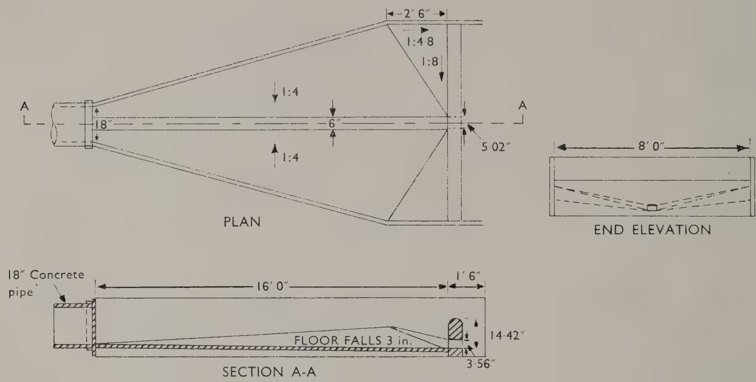


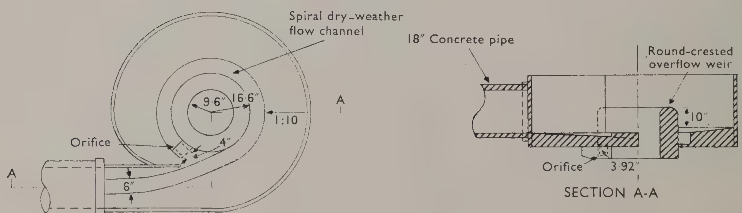
Fig. 19. DETAILS OF FIELD-SCALE MODEL OVERFLOWS.



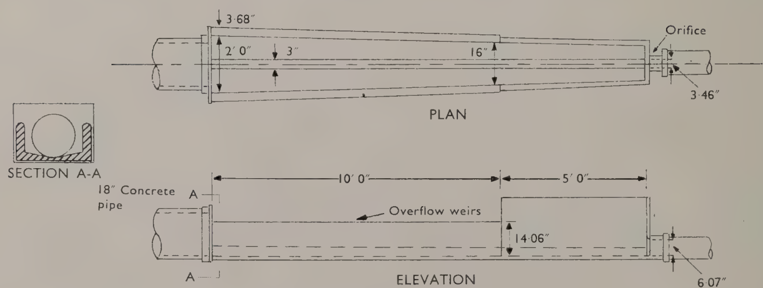
LOW SIDE – WEIR OVERFLOW



STILLING POND OVERFLOW



VORTEX OVERFLOW WITH CENTRAL SPILL

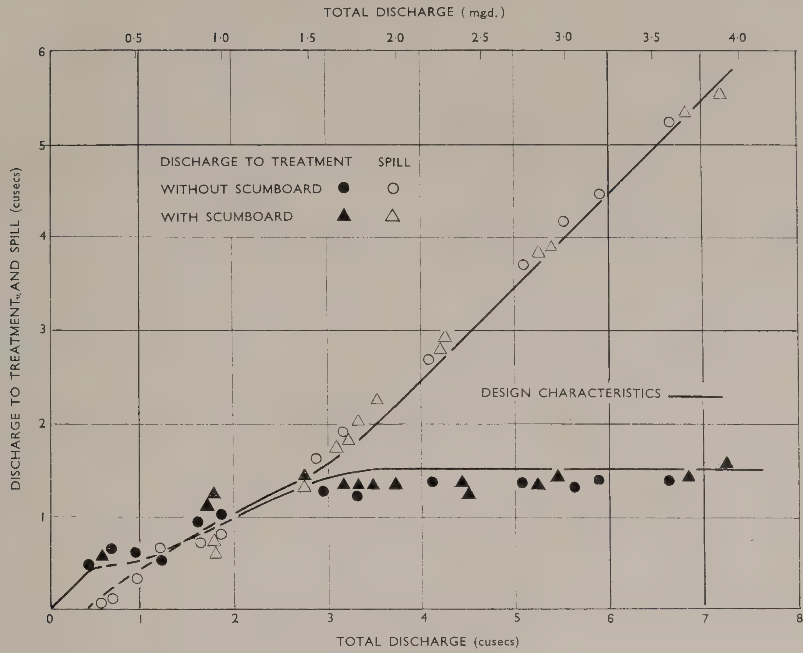


HIGH SIDE – WEIR OVERFLOW

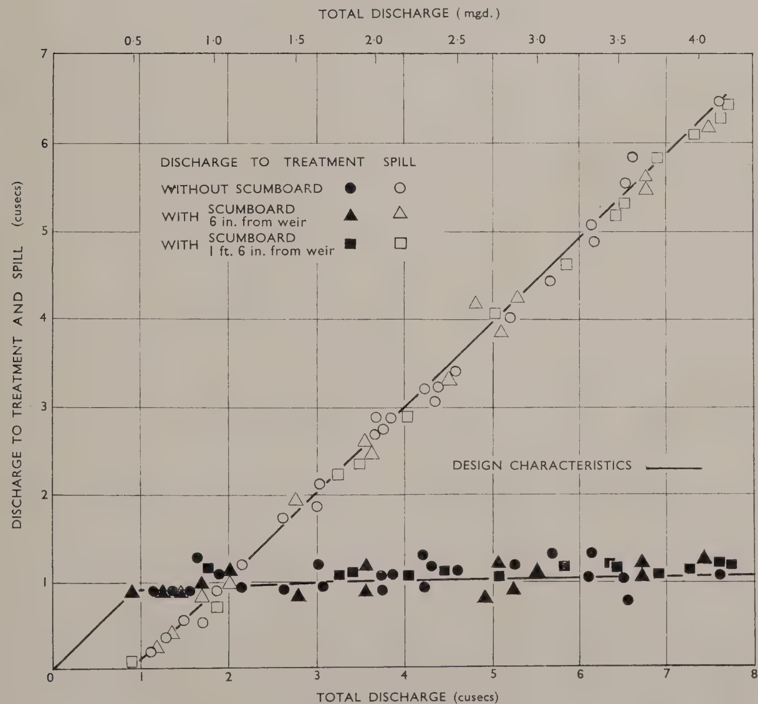
SCALE

1 0 1 2 3 feet

Fig. 20(a) DISCHARGE CHARACTERISTICS.
FIELD–SCALE MODEL OVERFLOWS.

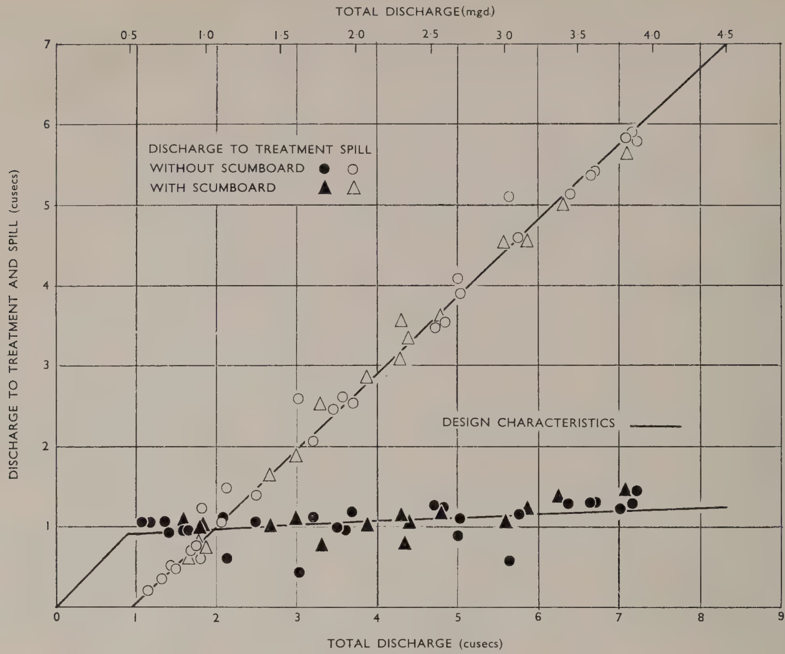


LOW SIDE – WEIR OVERFLOW

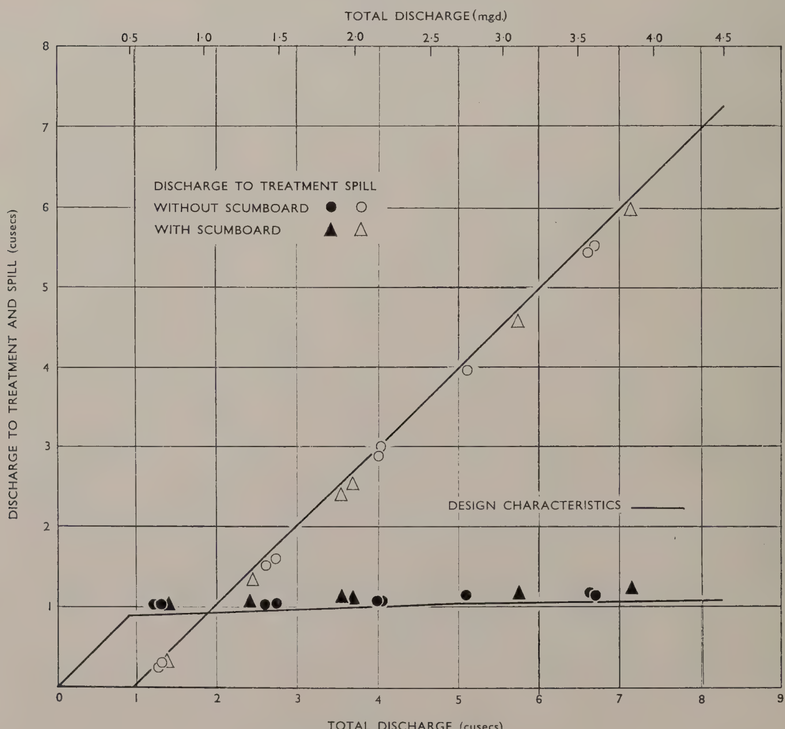


STILLING POND OVERFLOW

Fig. 20(b) DISCHARGE CHARACTERISTICS.
FIELD-SCALE MODEL OVERFLOWS.



VORTEX OVERFLOW



HIGH SIDE – WEIR OVERFLOW

effect of introducing a scum-board was to increase slightly the flow to treatment at all incoming flows, the increase being greater with the scum-board 6 in from the weir. There was a circulatory movement of the sewage in the fan-shaped chamber, with high velocities against the weir on one side and a back-flow on the other. This may have appreciably reduced the settling efficiency of the chamber.

Vortex The discharge varied slightly but consistently from the theoretical figures, due, it was thought, to the effect of circulation in the chamber. The water levels around the outer perimeter were somewhat higher than predicted, especially at high rates of flow. Introduction of a scum-board had little effect on the division of flow. There was a tendency for the orifice to become partly blocked by rags.

High side-weir Flow through the trough between the weirs was considerably slower and more tranquil than in the low side-weir overflow and spill took place over the full length of the weirs. Scum-boards had no effect on the division of flow.

Composition

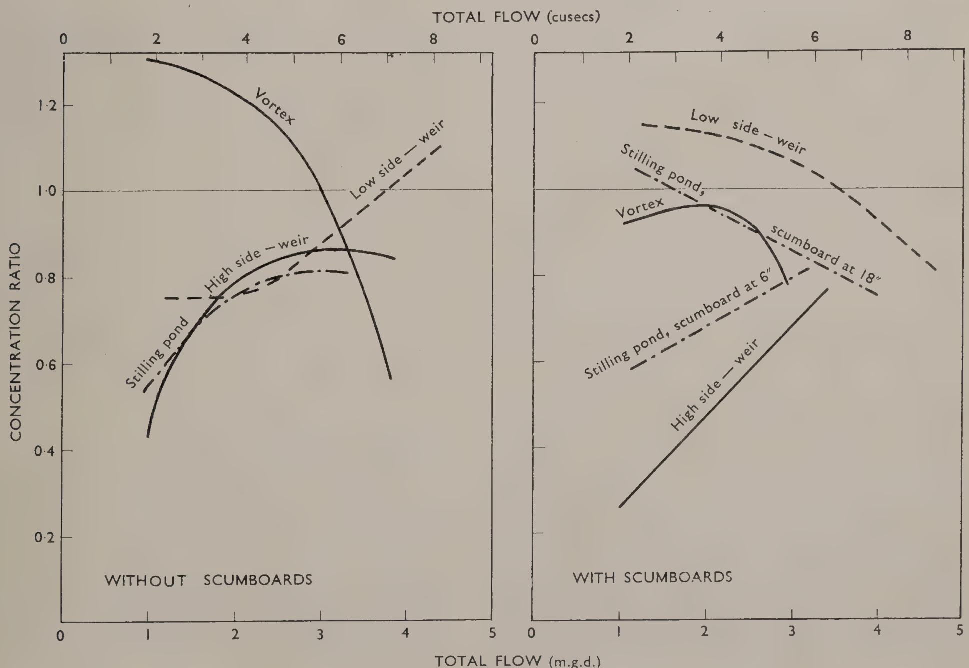
189. Although individual results showed some variation, there was no evidence in any overflows tested that, after screening, there was any consistent difference in strength between the flow to treatment and the storm sewage discharged over the weirs. The strength of the sewage was not significantly affected by removing gross solids. These represented only a very small percentage of the total polluting load.

190. The screenings collected from the flow to treatment and spilled flow during a period were weighed and expressed in terms of lb/mil gal. The ratio of lb/mil gal in the overflowed sewage to lb/mil gal in the flow to treatment (which we shall call the "concentration ratio") gives an indication of the effectiveness of the overflow in respect of discharge of gross solids. A ratio less than unity would indicate a tendency (which is desirable) for screenable material to pass to treatment rather than over the weir and a ratio greater than unity would indicate the reverse and undesirable tendency. There was a considerable scatter in the results, but the comparative performance of the overflows is shown in Fig. 21.

191. In the absence of scum-boards, the stilling pond and high side-weirs were the only overflows to show concentration ratios of less than unity at all flows. In all cases except the vortex, performance generally deteriorated as the flow increased. The curve for the vortex was, however, prepared from only a single run, so it is less reliable.

192. The use of scum-boards consistently improved the vortex and high side-weir overflows. The stilling pond performed better with the scum-board 6 in from the weir (the floor shape was designed for the scum-board at this position). With the scum-board 18 in from the weir, its performance was no better than an overflow without a scum-board. The best overall performance was that of the high side-weir with scum-boards.

Fig. 21. COMPARISON OF CONCENTRATION RATIOS.



193. Of all the overflows, the high side-weir with scum-boards showed the greatest tendency to pass faeces to treatment whereas the vortex tended to pass them over the weir. The stilling pond showed a tendency similar to the vortex, but less marked.

Summary

Note: The opening note (para. 173) in the Summary to Chapter 4 applies equally to these experimental models.

194. The low side-weir was shown to be inefficient as a device for accurate hydraulic separation. The weir length could have been substantially reduced without materially affecting the performance. It spilled prematurely and lacked control at high flows. There was a tendency for the hydraulic performance to be upset by rags and debris wedging at the downstream ends of the scum-boards. At all but the highest flows, the overflow without scum-boards tended to pass gross solids to treatment rather than over the weir, but no consistent improvement was noticed when scum-boards were provided. At lower flows, scum-boards appeared to have a detrimental effect.

195. The stilling pond with orifice control was satisfactory in its control of flow to treatment. Currents set up in the fan-shaped chamber, however, were not conducive to efficient settlement of solids, although a limited amount of separation was observed. In respect of retaining gross solids, the overflow without a scum-board was better than the low side-weir. Introduction of a scum-board seemed to produce little if any improvement and when moved from its designed position it appeared to have a detrimental effect. The overflow had a tendency to discharge faeces over the weir.

196. The vortex with orifice control, although not behaving entirely as predicted, was reasonably satisfactory in its control of flow to treatment. Its performance could probably have been improved had the design taken fuller account of the factors which affected its discharge characteristics. Its performance in retaining gross solids was poor except at high flows, but was generally improved by provision of a scum-board. It had a tendency to discharge faeces over the weir. (Research elsewhere[2] suggests that modifications to the geometry, coupled with a general increase in dimensions, would improve separating characteristics.)

197. The high side-weir with orifice control was comparable to the stilling pond in hydraulic performance. In this respect it was so superior to the low side-weir as to suggest conversion of low side-weirs to high side-weirs, coupled with outlet control, as being a cheap expedient to improve hydraulic performance of existing overflows on sewers laid at sub-critical gradients. (It would, of course, be necessary to ensure that such a modification did not cause unacceptable conditions of surcharge upstream.) With scum-boards, the high side-weir had the best performance of all the overflows tested in retaining gross solids and faeces.

198. Gross solids represented only a very small proportion of the total pollution passed over the overflows and there was no evidence that, after screening, the strength of the storm sewage discharged by any of the overflows tested was consistently different from the strength of the flow to treatment.

199. The experiments suggest that there is scope for further studies to compare the high side-weir overflow with stilling-pond overflows of various shapes. On the evidence of these experiments, the choice of the most efficient overflow appears to lie between these two.

200. All the overflows with the exception of the low side-weir would become more efficient at separating coarse solids if they were made very much larger in relation to the flow. It is unlikely, however, that this would significantly affect the division of polluting load which is not amenable to hydraulic separation.

Comparison of tests on laboratory scale and on field scale

201. The experiments at Luton were very different from those carried out in the laboratory at the Hydraulics Research Station and hence the results from the two series of tests are in many respects not comparable. All the laboratory runs were under conditions of varying flow and thus the effects of storage at the overflow and in the pipe upstream influenced the spill of the various pollutants that were simulated. This was not the case with the Luton tests, but on the other hand, these had the advantage of using real sewage. Also, only three of the four types of overflows tested at each place were common to both.

202. The Luton results indicated that none of the overflows tested there had any significant effect on the concentration of pollution in the discharge, but the laboratory results clearly demonstrate that storage and control characteristics will modify this conclusion in the practical situation of time-dependent flow. In this respect the results are complementary to each other.

203. Both series of experiments confirm how much better a stilling pond with outlet control is than a low side-weir in limiting the flow going to treatment. Both show the stilling-pond (when fitted with a scum board) to have advantages in handling gross solids. Neither suggests that the particular vortex overflow tested should be specially recommended. In these respects the two series of tests are confirmatory.

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CHAPTER 6. THE SETTING OF STORM OVERFLOWS

204. The general formula which has been employed for decades for the setting of overflows on sewers has been six times dry-weather flow (6 DWF). This has been criticised as not being scientific in derivation and as not defining precisely what is meant by DWF. It has also been criticised as permitting too much storm sewage to overflow and thus causing too much pollution.

205. In our Interim Report we made it clear that we ourselves shared the first of these criticisms, and that we would not recommend the perpetuation of a formula which was simply a multiple of DWF. It seemed to us, and we are still of the same opinion, that there was no logical reason why the amount of storm water to be retained in the sewer should be proportional to the dry-weather flow in that sewer. There was no reason why a community having a high demand for water, which resulted in a high DWF in the sewer, should, for that reason, be compelled to provide accommodation in the sewer and at treatment works for a correspondingly large volume of storm water. It seemed much more logical to have a formula in which the DWF of the sewage was indeed one term, but to have another term, a run-off term, to be added to the DWF and not to provide a multiple for it. Such a term would also largely avoid the problems arising from the difficulty of defining exactly what is meant by DWF—is it the dry-weather flow of sewage alone, should it include infiltration water, should it include industrial effluent, should it be the average flow or the maximum rate of flow in dry weather? If the expression for the setting is composed of the sum of two terms, one being the DWF and the other a figure which must be much greater than DWF, then the exact value of DWF is not going to matter a great deal and there is less need for its precise definition. It may be taken, however, that when we speak of DWF we mean the average daily rate in dry weather and not the maximum rate, and infiltration and industrial effluents are included in it. We must now consider what form the second term should take.

206. In the absence of infiltration and industrial effluent, the domestic sewage flow in dry weather is closely related to the domestic water consumption. In the traditional formula the overflow setting can then be expressed as six times the product of the population (P) and the average domestic water consumption in gallons per head per day (G), and written as $6PG$ gallons per day. This can readily be modified so as to make it much less dependent on the value of G by expressing the setting as $PG+PK$ where K is a constant. The term PK represents the amount of storm water retained in the sewer and is independent of the domestic water consumption. Such a formula would be simple to use, once a suitable value for K had been determined; it would be an improvement on the traditional formula and, we believe, the simplest improvement possible.

207. In considering what value of K would be suitable, we admit that we know of no method of calculation, by this or any other formula, which will give the right answer in every case. In fact, there might well be disagreement on what the right answer should be. We accept, however, as did those who had a hand in deciding on the existing formula, that the right answer must have due regard to costs. It must not be such that the costs of achieving it are out of all proportion to the benefits gained. Neither must it result in an overflow setting so low that the benefits obtained by other expenditure (such as on sewage purification in dry weather) are not fully realised (for instance because of fish kills due to excessive storm-sewage discharges).

208. We have not, in fact, carried out any cost-benefit studies on this matter, and do not know of anyone who yet knows in detail how they can be done; in any case, we would question whether we are in fact solely concerned with benefits which can be assessed in monetary terms, even though we are aware that progress has been made in the economic evaluation of amenity. We realise, therefore, that whatever formula we produce, there must be a stage in its derivation at which judgment as distinct from proven fact plays a part. This judgment may be either our own or that of people who have supplied us with information.

209. In Chapter 2, we reported upon a survey of some hundreds of existing overflows, as to whether or not they were regarded as satisfactory, and, if not, as to the reason for dissatisfaction. The judgment here is that of the river boards and must therefore be given considerable weight. It will be recalled that many overflows were reported as unsatisfactory because of their effect as one of a group, and although this indicated that the average setting was considered for some reason to be too low, it gave no information on the status of individual overflows within the group. In many instances, however, individual overflows were regarded as unsatisfactory and some of these were set above 6 DWF; in other instances, the reverse was the case and overflows set below 6 DWF were regarded as satisfactory. Clearly, therefore, factors other than the setting of the overflow affected the judgment. Nevertheless, the survey showed a general tendency for overflows set below 6 DWF to be reported as unsatisfactory and those at 6 DWF or over to be regarded as satisfactory. It may be that these opinions were influenced by uncritical acceptance of the traditional formula, but the fact remains that the survey gave little evidence which would justify our recommending a radical improvement over the old standards at the present time, bearing in mind that higher settings would generally entail increased costs both of sewers and of treatment works and that more benefit might be obtained by spending this money on other aspects of sewerage and sewage treatment.

210. We now consider the factor K. Domestic water consumption is often taken to be between 30 and 40 gallons per head per day (g.h.d.) so that, in the absence of infiltration or industrial effluents, similar results to the traditional 6 DWF setting could be obtained by fixing K, in the average case, at between 150 and 200 g.h.d. However, such a setting would merely maintain the status quo and we are of the opinion that settings higher than those represented by the old formula can be justified in some circumstances and will be increasingly desirable in the future. Since new works must be planned to take account of future conditions, it is our view that a normal requirement at the present time should represent a modest improvement over the old 6 DWF. In the absence of any rational basis, the extent of such improvement must remain a matter of judgment.

211. We are seeking a value of K higher than 200 g.h.d. and it should be rounded-off to avoid giving the impression that it is precise. The figures which come to mind are 250 and 300. It so happens that the mean of these—275—is very nearly $1\frac{1}{4}$ cubic metres, and with future metrication in mind, that was the figure we at first thought would be suitable. On balance, however, we consider the figure of 300 to be more appropriate in the long term. In the simple case of domestic sewage only, therefore, the setting (or the flow to treatment) would be expressed as

Setting $(Q) = PG + 300P$ gallons per day (g.p.d.) (9)

212. It is clear that this formula completely ignores (as does the traditional one) many factors which should ideally be taken into account. For example, it is obvious that allowance must be made for the presence of infiltration water and industrial effluents and it might be expected that the "right" setting of an overflow would also be affected by at least the following:

(a) Rainfall—either total, or frequency of occasions when a certain intensity is exceeded.

(b) The impermeable area draining to the sewer—an abnormally large area per head would seem to require an abnormally high setting.

(c) Time of concentration—a long time of concentration might reduce many peak flows which might otherwise have to be provided for, and might allow the flow in the receiving stream to respond to the storm before overflow takes place.

(d) The flow in the receiving stream—exceptionally high or low stream flows might permit lower or require higher settings.

(e) The use to which the stream is put.

(f) The situation of the overflow—if open to public view, a higher setting might be called for.

213. We do not have the information to enable all these factors to be taken into account quantitatively, but we would be remiss if we did not explore the possibility of taking some of them into consideration, either now or as the nature and magnitude of their effects become established.

214. It is easy to take account of infiltration which is ignored in the above formula (the term PG being the domestic sewage flow only). If infiltration water can be regarded as unpolluted—which it normally can—then the concentration of pollution from a storm overflow will not be increased if infiltration occurs. However, the overflow will operate more frequently because the setting will be reached with less rainwater present, so that, in all, the occurrence of infiltration water will increase the total pollution discharged over the overflow. This, in our view, should not happen; subsoil water should not occupy capacity in the sewer intended for rainwater. This is readily catered for by adding infiltration, I (gallons per day), to the formula, making it

$$\text{Setting } (Q) = PG + I + 300P \text{ g.p.d.} \qquad (10)$$

215. It is not so easy to include a term to take account of industrial effluents. If the total daily quantity were simply added to the last formula, then their presence would not alter the total period of overflow, but the concentration of the overflowing liquid and the total amount of pollution would certainly be increased and probably greatly increased. It is clear that this would rarely be satisfactory, and that the setting should be increased to reduce that pollution. However, as soon as the setting is increased by an amount greater than the daily industrial-effluent figure, the overflow will not come into operation so soon and the total period of operation will be reduced. It is impossible to devise a correction to the setting intended to accommodate industrial effluents and end up with conditions at all stages identical to those existing before the industrial effluent was present. It is even impossible to arrive at a practicable setting which will not, in the worst storm, permit more pollution to overflow than would occur with domestic sewage alone at the standard setting. There must be a compromise, in which the duration of overflow is reduced and the pollution discharged sometimes reduced to zero and sometimes increased.

216. We have little information to help us in judging the compromise, but simple calculation suggests that, unless the industrial effluents are several times as strong as domestic sewage, it would usually be satisfactory to provide for two volumes of rainwater for each volume of industrial effluent present to be taken to the works for treatment. Thus if E is the volume of industrial effluent discharged in 24 hours, the setting would become

$$Q = PG + I + E + 300P + 2E \text{ g.p.d.} \qquad (11)$$

The term "$PG+I+E$" is the dry-weather flow in the sewer (DWF) and so we get

$$\text{Setting } (Q) = DWF + 300P + 2E \text{ g.p.d. } \textit{Formula A} \qquad (12)$$

217. In our considerations, we have assumed that the whole area draining to the overflow is sewered on the combined or partially-separate systems and P is the population of this area. It is fairly common, however, for a development on the separate system to be connected to the sewers of a "combined" area and it is necessary to examine what allowance should be made in the setting for this.

218. If the setting were raised so that 1 DWF from the "separate" area was carried forward to treatment in addition to the amount stipulated by *Formula A*, the period of overflow would, in theory, not be altered, but the overflowing storm sewage would be stronger because the proportion of sewage to rainwater would be increased. This is one extreme and would be unsatisfactory. However, as soon as the setting is increased by an amount greater than 1 DWF from the "separate" area, the total period of overflow will be reduced—a situation similar to that which we discussed above in dealing with industrial effluents.

219. As with industrial effluents, it will not be possible to ensure that, with a "separate" area connected, the conditions will always be the same as with a combined system alone. A compromise is necessary lying between, on the one extreme, the addition of 1 DWF and, on the other extreme, the full allowance that would be needed to give the same dilution as would occur in a combined system alone. Since, however, it is normal to provide full-treatment units for flows up to 3 DWF (the definition of 3 DWF in the context of sewage treatment is discussed in para. 303), it is reasonable to assume that the accommodation that should be provided for the "separate" area should be not less than this. Some fairly simple calculations demonstrate that this figure is, in fact, a reasonable compromise and it is the allowance we suggest should normally be adopted for design.

220. If the separate system is expected to receive, either by design or through illicit connections, significant amounts of surface water—as, for example, in the case of polluting surface water from an industrial area—special consideration would need to be given, and in an extreme situation it might be necessary to treat the "separate" area as though it were drained on the combined system. From the point of view of good practice, it would be better, if reasonably practicable, to connect any "separate" areas to the "combined" sewer downstream of the lowest storm overflow.

221. We have in mind that where the circumstances are particularly favourable (e.g. overflows into large rivers that are not over-polluted), a figure somewhat lower than 300 g.h.d. would be appropriate. Conversely, in other situations (e.g. where streams are small or sluggish), a higher figure might be desirable. Similarly, abnormally high quantities of strong industrial effluent might call for an increase in the term $2E$. Such departures from the normal value would need to be justified on the merits of the case, but considering the way in which the figure of 300 g.h.d. has been arrived at, it would clearly not be right to try to justify small variations to suit circumstances. We would regard 50 g.h.d. as being a sensible minimum variation.

222. *Formula A* is relatively simple, it is easy to calculate, it is related to some formulae which have been used but is nevertheless an improvement upon them, and the terms in it are such that, on the evidence of the survey described in Chapter 2, its adoption would maintain or bring about satisfactory conditions in most cases. It certainly has a claim to be discussed with other formulae later in this chapter. But it still ignores many of the factors listed earlier. For instance, it would appear that the impermeable area draining to the overflow ought to be taken into account in the calculation of the required setting. The formula we have been considering does not do this or, at most, it makes the assumption that the impermeable area per head of population is a constant. However, our research has confirmed that the amount of storm water reaching the sewer is almost directly proportional to the impermeable area.

223. We can express the flow in the sewer as the sum of two terms as before, thus

$$Q = DWF + kA \qquad (13)$$

where A is the impermeable area and k is a new constant. Since flow has the dimensions of volume per unit time, and A is an area, k must have the dimensions of length per unit time. It is, in fact, a measure of rainfall intensity—not, however, the actual intensity at the time in question, but rather the steady rainfall intensity which, together with the dry-weather flow, would give rise to the steady flow Q arriving at the overflow if there were 100 per cent run-off from the impermeable area and none from the remainder. (As a close approximation, the term kA will be in cusecs if A is in acres and k is in inches per hour.) If we now decide on the critical value, i_C, of the effective rainfall intensity, above which overflow could be permitted, the setting could be expressed as

$$\text{Setting } (Q) = DWF + 0.543\, A i_C \text{ m.g.d.} \qquad (14)$$

when DWF is in m.g.d., A is in acres and i_C in inches per hour (in/h).

224. In our experimental work at Northampton and Brighouse and, with reservations, at Bradford, it was found that an empirical relationship existed (Equation 2, para. 88) between the number of hours that a given flow of storm sewage was exceeded, the total rainfall and the effective rainfall intensity; application of this equation also gave good agreement with observed results from a partially-separate system at Luton. If the permitted number of hours of spill per year is T and the annual rainfall is R inches, then (from Equation 3, para. 91)

$$i_C = 0.1135 \left\{ \left(\frac{20R}{T} \right)^{\frac{1}{4}} - 1 \right\} \text{ in/h.} \qquad (15)$$

Converting in/h to m.g.d./acre and substituting in the equation for the overflow setting, we obtain (from Equation 4, para. 91)

$$\text{Setting } (Q) = DWF + 0.0617\, A \left\{ \left(\frac{20R}{T} \right)^{\frac{1}{4}} - 1 \right\} \text{ m.g.d.}$$

Formula B (16)

Given the appropriate value for T, the formula is quite workable, and would not involve getting any more information about an area than an engineer would normally need to know.

225. Critical appraisal of *Formula B* will inevitably take more space than that of *Formula A*, for which all we needed to say was that it took no account whatever

of a number of the factors we have listed. *Formula B* takes account of two of these factors—rainfall and impermeable area—but it is necessary to examine whether they have been taken into account properly, and whether any appreciable improvement has been brought about.

226. The application of *Formula B* requires a value for T, the average permissible number of hours of overflow per annum, but this term (unlike R and A which are measurable) is a matter of judgment, and we feel that we have too little information on which to decide a suitable value. The difficulty did not arise to the same extent in fixing the figure of 300 in *Formula A*, because it is of the same type as the traditional formula and we were able to relate the figure to existing overflows about which we had the opinions of the river boards. We do not, however, have any information about the hours of operation of these existing overflows.

227. The only way of selecting a value of i_C appears, therefore, to be on the basis of the experimental work from which *Formula B* was derived, and although we have no reason to suspect that the sites studied were atypical, we do not know how near the average they are. On the basis of the typical figures given in Table 7 (para. 93), 100 hours per annum was suggested as a possible standard for T; the derivation of this value took account of one of the aims of *Formula A* in that it was intended to raise the average setting somewhat above 6 DWF. We also considered that there might be reasons for suggesting that the magnitude of T should be different for different circumstances, but there was insufficient evidence on which to decide on a single value for T, let alone on a range of values.

228. In the allowance made for rainfall it is assumed that the permissible number of hours of operation is independent of the rainfall, so that higher rainfall requires higher settings. At first sight this seems reasonable, and pushed to extremes is plainly true, but some of us have serious doubts whether higher settings are needed in areas of higher rainfall in our country with its medium rainfalls. They argue that, in general, our wetter areas are those where soil tends to be less permeable and where gradients are steeper, and that, for both reasons, river levels rise more rapidly after the onset of rain and are thus better able to cope with storm overflows than are rivers the rise of which is greatly delayed. Hence, they hold, there is no need for higher settings in the wetter areas. We may add that we have not heard of any river authority requiring higher-than-normal settings solely because of higher rainfall.

229. If it is concluded that rainfall should not affect the determination of the setting, then, for a given value of T, *Formula B* reduces to the form from which it was derived, namely $Q = DWF + kA$ (Equation 13, para. 223), which takes account of impermeable areas but ignores population. *Formula A* has a similar basic form (para. 206), but it takes account of population and ignores impermeable areas. To examine the relative merits of these two methods we shall consider what conclusions they lead to when applied to densely populated areas, or, to be more precise, areas with impermeable areas per head that are well below the average.

230. *Formula A* would give the same proportion of sewage and surface water in the sewer at first spill in a densely as in a sparsely populated area, but in the former case the overflow would operate less frequently, and the polluting load discharged per head of population would be lower. Use of *Formula B*, on the other hand, would result in the frequency of operation being the same in both areas, but at first spill the surface water would be a smaller proportion of the total flow in the dense area, so that the strength of the discharge would be greater, and the polluting load discharged per head of population higher.

231. We are agreed that the result given by *Formula B* is not what is wanted—the polluting load per head from a densely populated area should not be greater than the norm. We are not entirely satisfied with the result given by *Formula A*, but are agreed that it is a better alternative. As densely populated areas are normally also areas of high total population, it may well be argued that the control of discharges of storm sewage should be more stringent and thus that, qualitatively, *Formula A* is acceptable in this respect—qualitatively because we have not the data from which we can say quantitatively how the polluting load per head will change with increasing density of population, nor are we prepared to say how it ought to vary. We are thus left with the conclusion that, by ensuring less frequent discharge (and a smaller total discharge) from the more densely populated areas, *Formula A* is preferable to *Formula B*.

232. In this discussion we have been involved in the difference between two aims, namely, limiting the hours of operation of the overflow, and limiting the amount of polluting matter discharged. We have not previously had to distinguish between these, and our survey of overflows did not permit this to be done. It is true that in that survey many overflows were described as unsatisfactory because they operated too frequently, but in most cases it could be inferred that the statement was made merely because the setting was below 6 DWF. However, there were also many cases where the overflow was unsatisfactory in that it resulted in pollution of the stream and even in fish deaths, in which occurrences it is the amount of polluting matter that is the more important.

233. We are agreed that it is insufficient to decide upon a setting solely on the basis of a formula which limits the average number of hours of operation per annum and ignores any dilution which may occur before the overflow begins to operate. This does not necessarily mean rejecting *Formula B*; it may merely need combining with *Formula A* or, alternatively, it may need supplementing by another which deals with the pollution aspect. A particular overflow would then have to satisfy both formulae.

234. Any formula for limiting the amount of pollution discharged must bring in quality as well as

quantity, and since the principal reason for limiting the amount of pollution is to protect the quality of the stream (using the word quality in the widest sense and including in it visible signs of sewage), then ideally the flow and quality of the receiving stream should also be taken into account. If the flow in the stream were known and its quality (for instance, on a BOD basis) could be assumed, and if the BOD of storm sewage could also be assumed, then a formula could be worked out for a setting at which the total spill would never (or very rarely) raise the BOD of the river above a figure which could be agreed as being tolerable.

235. Unfortunately, though a lot of information exists on the effect of BOD on the condition of a river, this is all in relation to a continuous load. There is good reason to expect that the response of a river to a highly intermittent load would be very different. We endeavoured to find a site where we could investigate this point, but as mentioned in Chapter 1, were not successful. Thus we cannot complete such a formula, because we do not know what should be the acceptable BOD; furthermore, it would be unrealistic to presume that the acceptable BOD would not vary from river to river—depending on the depth, velocity, use and so on. Finally, we have no information by which we could relate such a formula to present practice.

236. With the development of a wide range of recording analytical instruments, it should be possible before long to get much more information about pollution caused by storm discharges, and eventually it may be possible to devise some reliable method of taking pollution into account in a formula giving the setting. We hope that an attempt will be made to reach this goal, but meanwhile we cannot suggest any method which deliberately and quantitatively takes into account river quality, character and use.

237. There remains the possibility of relating the permitted overflow to the permitted discharge of final effluent. It may be argued, for instance, that when river conditions call for a very high-quality effluent in dry weather, then a correspondingly high setting of storm overflows would be justified. The Memorandum on Effluent Standards[1] also draws attention to this possibility.

238. It must be accepted that, unless overflows are to be set very much higher than they are now, there will inevitably be times when the rate of discharge of pollution from them will be much greater than from treated effluents discharged at the same time. For example, if the flow arriving at the overflow is more than twice that at first spill (by no means an uncommon occurrence), more than half the flow will be discharged with no treatment at all, whereas, in dry weather, all will be treated to reduce its BOD by perhaps 95 per cent. The fact that such intermittent storm-sewage discharges rarely appear to cause serious trouble is one of the reasons for believing that the response of the river to spasmodic pollution is very different from its response to continuous pollution. Nevertheless, it is not unreasonable in principle to suppose that the permitted storm-overflow discharge should bear some relation to the permitted dry-weather discharge, and the following arguments attempt to bring this into a formula.

239. Our investigations at Northampton, Brighouse and Bradford were discussed in Chapter 3. It will be recalled that, when the volume (expressed as a percentage of the contributory rainfall) in excess of a particular flow (expressed as effective rainfall intensity) was plotted against this flow, reasonably smooth curves resulted (Fig. 7); those for Northampton and Brighouse were almost coincident up to 0.10 in/h and remained very close together to about 0.15 in/h. The curve that fitted the data best was given by Equation 6 (para. 99),

$$v = 75\,(1+6i)^{-4.4} \text{ per cent.} \qquad (17)$$

This may be regarded as the volume which would be discharged from an overflow set at i in/h, and it may be noted that when $i = 0$ (i.e. when all the run-off is is allowed to discharge) $v = 75$ per cent, so that the run-off constitutes three-quarters of the rainfall on the impermeable area. The curve for Bradford depends on whether the data relating to the inexplicably long periods of run-off are included. If they are, then the curve is on one side of the Northampton–Brighouse curve; if they are not, it is on the other side by an almost equal amount. It is thus not really possible to say whether the Bradford results tend to support or refute those from the other two sites, but comparison with records from a fourth site, at Luton, gave good agreement over the same range of effective rainfall intensity (Fig. 8).

240. The above formula is readily converted to more familiar terms of quantity spilled (V mil gal per year) by bringing in the total rainfall (R inches per year) and the impermeable area (A acres), together with the numerical factor relating acre-inches to million gallons.

This gives

$$V = 0.017\,A\,R\,(1+6i)^{-4.4} \text{ mil gal/year.} \qquad (18)$$

To convert to terms of quantity of pollution, the volume must be multiplied by the strength of the storm sewage discharged. Storm sewage is by its very nature extremely variable in composition, but if we take BOD as the criterion of quality, and this is probably the most suitable one in the circumstances, then on broad overall average it was found that the strength of the storm sewage at Northampton, Brighouse and Bradford was about 30 per cent of that of the average crude sewage. Taking the BOD of the crude sewage (including infiltration water and industrial effluents) to be B mg/l, the estimated annual load (L_s) discharged as storm sewage becomes $V \times 0.3\,B \times 10$ lb/year or

$$L_s = 0.051\,A\,R\,B\,(1+6i)^{-4.4} \text{ lb/year.} \qquad (19)$$

If, in dry weather, the total flow passing down the sewer is DWF m.g.d., and if the treatment provided reduces the BOD of this sewage to y per cent of B, the corresponding annual BOD load (L_e) of the sewage effluent is $365 \times DWF \times B\,y/100 \times 10$ lb/year or

$$L_e = 36.5\,B\,y\,DWF \text{ lb/year.} \qquad (20)$$

241. Although the difference in effect between continuous and intermittent pollution has been pointed out, it seems reasonable to suggest that the annual total permitted BOD load from an overflow should not exceed the load from the treated effluent originating in the sewage passing that overflow in dry weather. Suppose that the permitted load from the overflow is to be z per cent of that of the treated effluent, then the critical effective rainfall, i_c in/h, which now replaces i, is found by putting L_s equal to z per cent of L_e. On rearranging, this gives

$$i_c = \frac{1}{6}\left\{\left(\frac{0.14\,AR}{yz\,DWF}\right)^{\frac{1}{4.4}} - 1\right\}\text{in/h.} \tag{21}$$

Substituting in the earlier general formula of Equation 14 (para. 223) then gives

$$\text{Setting } (Q) = DWF + 0.09A\left\{\left(\frac{0.14\,AR}{yz\,DWF}\right)^{\frac{1}{4.4}} - 1\right\}\text{m.g.d.} \qquad \textit{Formula C} \quad (22)$$

242. This formula, too, is quite workable, and might be used independently of, or in conjunction with, *Formula B* which controls the duration of spill. If used in conjunction, the recommended setting would be the higher of the two calculated values. However, *Formula C* cannot be used until a value has been assigned to z, the permitted annual BOD load discharged as storm sewage, expressed as a percentage of the equivalent load discharged as sewage effluent.

243. *Formula C* can be re-arranged to give

$$z = \frac{0.14\,AR}{y\,DWF\left(1 + \frac{Q - DWF}{0.09A}\right)^{4.4}} \text{ per cent} \tag{23}$$

and it is of interest to see what would have been the values of z at the three sites, assuming values of y based on a sewage works effluent with a BOD of 20 mg/l and presuming accurate control of the overflows. The values of z, based on these assumptions, are found to be

	For $Q \equiv 6DWF$	*For* $Q \equiv 9DWF$
Northampton	101%	72%
Brighouse	9%	4%
Bradford	65%	42%

If the overflows were set according to the suggested *Formula A*, the corresponding values of z would be 65%, 15% and 34%.

244. The merits and shortcomings of *Formulae A* and *B* have already been discussed; it is now necessary to consider *Formula C* in the same way.

245. In the first place, as with *Formula B*, it is based on the empirical expression for i which fits the results from three drainage areas quite well within the limits which are relevant although it does not fit the fourth one well. This agreement is encouraging in view of the very different natures of the drainage areas and sewerage systems studied; but all these systems had rather similar times of concentration, and we do not know how accurately the expression will fit areas with very different (and particularly with much longer) times of concentration. Secondly, it is based on the fact that, at the sites studied, the storm sewage had a BOD, on average, 30 per cent of that of the crude sewage, and on the assumption that this would be broadly true in general. How nearly true it is we do not know, but in the experimental areas it appeared to be fairly consistent for overflow settings between 3 and 9 DWF. It might not have been true for higher settings or for highly industrialised areas. Thirdly, and perhaps most important of all, we have little data from which to assign a value to z, although it is not unreasonable to suggest that it should not exceed 100 per cent. Finally, it is impossible at present to compare the settings calculated by *Formula C* with existing overflows known from our survey to be either satisfactory or unsatisfactory.

246. We have considered other possible formulae and, in particular, whether it would be worthwhile introducing a term which varied directly with the characteristics of the receiving stream and its flow in relation to the DWF in the sewer. The survey of overflows provided a small amount of practical evidence that this could create an improvement, but we have no idea or evidence of what the magnitude of such a term should be, so we have not pursued the matter further. It remains for us to make a choice from those already described.

Summary of considerations

247. We would have liked to have been able to recommend a formula for overflow settings that took account of river quality, flow and use. We did consider such a formula, but there were too many unknown factors to make it workable. We still consider this to be the ideal, but, although we hope that it will eventually be possible for such a formula to become the basis of overflow settings, it must be many years before it is a practical proposition.

248. The only evidence we have on the effect of storm-sewage discharges on river conditions is that which we obtained in the survey described in Chapter 2. We do not know the accuracy of the data supplied to us on sewage flows in dry weather and at first spill—our own experiments have shown us the difficulties of measuring these terms—and we have had to assume that any systematic error in the data is insufficient to invalidate our general conclusion that, while no very great changes in existing overflow settings are called for, the average setting should be raised somewhat above the present norm of 6 DWF.

249. We are agreed that it is inadequate simply to clarify the definition of DWF and to increase the multiple to some value greater than six; the principle itself requires changing, because inappropriate allowance is made for the effects of variations in water usage, for infiltration, and for industrial effluents. We have considered various formulae which attempt to achieve these two aims of raising the average setting and making appropriate allowance for the factors mentioned.

250. Perhaps the most rational method developed is *Formula C*, which sets out to relate the BOD load discharged from the overflow to that discharged in the treated sewage effluent. The logical conclusion would appear to be that this method would need to be applied not only to the individual overflows but also to the whole of the sewerage system; thus, for a given effluent load it would be reasonable to permit a greater average load per overflow if there were few overflows than if there were many. *Formula B* is somewhat simpler than *Formula C*, being intended only to limit the yearly duration of discharge, and hence it contains no term involving the composition of storm sewage. *Formula A* is simpler again and its scope is restricted to achieving a modest improvement on present practice and to making a more appropriate allowance for variations in water usage, infiltration and industrial flow than is made at present. *Formulae B* and *C* attempt to put the criterion for overflow settings on a more rational basis, and in so doing they introduce terms which are based on the experimental work carried out on behalf of the Committee. The extent to which the data used are representative of the sewerage systems in this country is, however, unknown, and becomes a matter of judgment.

251. It is clear that none of these formulae can possibly be precise in the sense that its application could control the amount or quality of overflow accurately to predetermined levels. Furthermore, none is likely to be fully applicable to areas with very long times of concentration. For example, in the extreme case of the London area, *Formula A* would probably give a setting equivalent to about 7 times the average flow in dry weather, whereas, in practice, the overflows come into operation at values ranging between about $2\frac{1}{4}$ and 6 times[2] and the total BOD load estimated to have been discharged from 26 overflows in 1952 was only 4 per cent of the equivalent yearly load of the sewage effluents from the whole system (this value could have become about 40 per cent for an effluent BOD of 20 mg/l).

252. We have already criticized each of the formulae we have proposed and there is no point in repeating these criticisms at length. Suffice it to say that we are almost unanimously in favour of *Formula A*, the principal reasons for this choice being that:

(a) since existing practice (when complied with) appears to give satisfaction far more often than not, and since *Formula A* is clearly an improvement on existing practice, there is no compelling urgency to ask for a still better formula;

(b) it is not established in practice that overflow settings should depend upon the local rainfall, and we are therefore not justified in adopting a formula which brings this in;

(c) the equations used in *Formulae B* and *C* are empirical and, although they satisfy two or three cases reasonably well, it is not clear that they will apply elsewhere;

(d) *Formula C* is intended to control pollution, but it does not take quality into account except to assume that the average strength of storm sewage in relation to crude sewage in the test areas will be the same as everywhere else. The validity of this assumption is doubtful;

(e) we have no data to relate *Formulae B* and *C* to present practice, which it is felt we must be able to do;

(f) it is arguable whether a frequency type of formula would ever be needed if a satisfactory pollution equation was available and was met.

On the other hand, a small minority of us feel that, having developed formulae such as *B* and *C*, further discussion could have overcome the remaining objections to their application. This minority put forward the following reasons against the adoption of *Formula A*:

(a) It is neither as rational nor as objective as some of the alternatives considered.

(b) It makes no direct use of the results of the research work carried out under the Committee's direction.

(c) It takes no account of the different characteristics of the drainage areas and receiving waters.

(d) Whereas *Formula C* takes account of the BOD of the whole crude sewage including infiltration water and industrial effluents, there seems little justification for the simple multiplication of the industrial-effluent flow in *Formula A*, when industrial effluents are so variable in quality.

(e) *Formula A* provides little scope for incorporating new evidence based on experience, river-quality data or further research, and fails to provide stimulus or guide lines for such development.

253. Nevertheless, the minority accept that an overriding consideration is that an unambiguous recommendation be given. They therefore accept that the opinion of the overwhelming majority must prevail, but suggest that formulae of the type of *B* and *C* should be subjected to further testing and development. It thus becomes possible to present a unanimous recommendation for the adoption of *Formula A* (para. 216). We realise, however, that there will need to be at least a few exceptions where there will be justification for increasing (or decreasing) the norm of 300 and for increasing the industrial-effluent multiplier. We have in mind cases where the stream conditions are such as to justify a particularly high (or low) standard of treated effluent, and cases where the industrial-effluent load constitutes an exceptionally high proportion of the sewage load.

254. To meet these circumstances, and in the interest of examining the relative merits of the various formulae, we further recommend that, in suitable cases, when the setting has been derived from *Formula A*, the values of T and of z should be estimated from Equation 5 (para. 92) and from Equation 23 (para. 243) respectively. Where exceptionally high or low values are found—such as z exceeding 100 per cent—

consideration should be given to whether there are reasons to suspect that *Formula A* is inapplicable in the particular case.

255. We do not expect that, immediately our report is published, there will be a sudden and urgent demand for all overflow settings to be re-adjusted on the new basis. Our survey showed that at least four out of every five individual overflows set according to previous standards were satisfactory, and there is no obvious reason why they should not be left as they are. However, as the overflows come up for reappraisal, and particularly if existing circumstances change, we hope the settings will be made to conform with what we now recommend.

References

1. Ministry of Housing and Local Government. Technical Problems of River Authorities and Sewage Disposal Authorities in Laying down and Complying with Limits of Quality for Effluents more Restrictive than those of the Royal Commission. (H.M.S.O. 1966).

2. Department of Scientific and Industrial Research. Water Pollution Research Technical Paper No. 11. Effects of polluting discharges on the Thames Estuary (p. 89). (H.M.S.O. 1964).

CHAPTER 7. STORM OVERFLOW STRUCTURES

Overflows in current use

256. In our survey of existing storm overflows described in Chapter 2, we sought information about the types of overflow in current use and this is summarized in Table 21. It will be recalled that, if we excluded from our consideration all overflows set at less than 6 DWF, together with the overflows that were classified as unsatisfactory solely because of their combined group effect, the number unsatisfactory was reduced from 317 (out of 849) to 106 (out of 603). The table shows how these are divided by type. It should be noted that the description of an overflow as unsatisfactory does not necessarily imply that the overflow is unsatisfactory because of its type. For example, an overflow which operated in dry weather would be expected to be unsatisfactory because of its setting, regardless of the type of structure.

257. It will be seen that the majority of existing overflows are of the side-weir type and that these are divided almost equally between high and low-weir types, each comprising about one-third of the overflows in the survey. The investigations at Bradford and Brighouse showed up the poor control characteristics of the low side-weir, resulting in flows much greater than the nominal setting being passed to treatment as the quantity overflowing increased. This was supported by the laboratory and field-scale experiments described in Chapters 4 and 5. Contrary to expectations in the light of these results, however, a much lower proportion of low-weir types was classified as unsatisfactory compared with high-weir types.

258. It might be inferred that many of the low-weir overflows were considered satisfactory because, overall, they retained a larger proportion of the storm sewage than a high-weir type of the same nominal setting would have done, and that consequently the low-weir type of overflow is to be preferred to the high-weir type for pollution control. However, it is necessary to point out that the ability of the structure to retain these higher flows must depend upon the capacity of the sewer downstream to accept them, and a low-weir with a nominal setting of, say, 6 DWF, coupled with a downstream-sewer capacity of, say, 12 DWF, could be replaced with a high-weir which, because of better control characteristics, could be set at, say, 10 DWF with obvious advantage to the river.

259. As would be expected, the percentage of the more modern types of overflow structure (siphon and stilling pond) included in the survey was small. The fact that a smaller proportion of them was classified as unsatisfactory may have been due, in part, to the fact that their settings were more appropriate to current sewage flows. It is also possible that more care was exercised in the selection of sites and points of discharge.

Design considerations

General

260. Ideally, a storm overflow should achieve the following:

(a) It should not come into operation until the prescribed flow is being passed to treatment.

(b) The flow to treatment should not increase significantly as the amount of overflowed storm sewage increases.

(c) The maximum amount of polluting material should be passed to treatment.

(d) The design should avoid any complication likely to lead to unreliable performance.

(e) The chamber should be so designed as to minimize turbulence and risk of blockage; it should be self-cleansing and require the minimum of attendance and maintenance.

TABLE 21. Summary of types of overflow covered by survey

Type of Overflow	All overflows in survey		Overflows set at 6 DWF or higher, excluding those unsatisfactory solely for combined effect	
	Number (Percentage of total in brackets)	Number unsatisfactory (Percentage of each type in brackets)	Number (Percentage of total in brackets)	Number unsatisfactory (Percentage of each type in brackets)
Pipe	131 (15)	34 (26)	107 (18)	17 (16)
Low side-weir	282 (33)	101 (36)	200 (33)	30 (15)
High side-weir	277 (33)	146 (53)	166 (27)	48 (29)
Siphon	11 (1)	1 (9)	10 (2)	— (—)
Stilling pond	42 (5)	6 (14)	37 (6)	2 (5)
Leaping weir	34 (4)	13 (38)	21 (4)	2 (10)
Miscellaneous	57 (7)	14 (25)	47 (8)	5 (11)
Unknown	15 (2)	2 (13)	15 (2)	2 (13)
Total	849	317 (37)	603	106 (17.5)

When considering these requirements, regard must be given to site limitations, accessibility and the need for overall economy.

261. It is clear that any attempt to reconcile these requirements must involve a large element of compromise. Furthermore, it is difficult to dissociate the requirements of a storm overflow from its setting. In general terms, the lower the setting the higher must be the standard of performance of the overflow, particularly in regard to hydraulic efficiency as well as to its ability to prevent the overflow of solid matter.

262. Hydraulic conditions at an overflow are complicated by changes in direction and velocity of flow, resulting in considerable turbulence, and performance is difficult to predict with certainty. Considerable energy is dissipated in accommodating these changes in direction and velocity, and this can lead to the establishment of hydraulic jumps or standing waves and interference from reflected waves, with a changing pattern under constantly changing flow conditions. These effects can be lessened by reducing velocities and changes of direction to a minimum; velocity reduction can only be achieved by enlarging the structure, but, apart from considerations of economy, it is necessary to ensure against the fouling of the chamber as a result of cumulative deposition of solid matter, and eventual interference with performance.

Practical minima

263. The two most significant factors affecting the design of overflow structures are hydraulic limitations and the handling of the wide variety of solid matter liable to be present in sewage. Both these factors influence the minimum practical dimensions of the outlet through which the flow to treatment must pass. The outlet is usually in the form of an orifice or throttle pipe, and the available hydraulic head for control purposes (to ensure that the maximum flow to treatment does not exceed by too great a margin the setting or flow at first spill) determines the size or cross-sectional area of the outlet. At the same time, the outlet must be of a shape and size which is capable of passing solid matter without becoming blocked. Professional opinion varies as to what is the minimum practical size for the outlet through which the flow to treatment passes, but we suggest that it should have a cross-sectional area of not less than 36 square inches (roughly equivalent to a 7-in diameter pipe) with a minimum dimension in any direction of 6 inches.

Control of flow and of gross solids

264. The design of an overflow chamber aimed at the achievement of quantity and solids control, solely by hydraulic design with no moving parts, cannot be expected to be precise, and the smaller the flows to be handled the lower are the prospects of success.

265. When large flows are involved there is greater scope for the introduction of mechanical aids, both for control of the flow passed to treatment and for the control of floating solids. Some form of throttling device is needed to control the flow to treatment, and, to maintain that flow at a constant rate, progressive throttling is called for as the rate of overflow increases.

266. If the structure is effective in preventing the overflow of floating solids, it follows that, if control is by a drowned orifice, these solids must accumulate until the flow to treatment falls to a level at which the outlet is no longer "drowned" and a free surface is available for floating matter to be carried forward. This is an aspect which should be borne in mind in design.

267. The larger items of solid matter in storm sewage such as faeces, contraceptives, paper and rags,although constituting an insignificant proportion of the polluting load, can become aesthetically offensive if they are discharged from an overflow and subsequently become stranded on the banks of a stream or are caught up on overhanging vegetation and are left dangling as the stream flow subsides after a storm. Attempts to retain these solids in the sewer by the installation of scum-boards, fixed screens of the bar type, or louvre-type screens designed to be self-cleansing, have met with indifferent success, their effectiveness often depending on their location relative to the overflow weir or on the geometry of the overflow chamber. Screens in particular need regular attention and, if neglected, may cause the overflow to become inoperative with consequent overload difficulties at sewage treatment works. In some cases they may even be the cause of local flooding and there are instances where such devices have been installed but have been subsequently removed because of the problems of inspection and maintenance. Yet the need to prevent damage to visual amenity resulting from the discharge of storm sewage containing gross solid matter has become increasingly important.

268. The introduction of power-operated mechanical equipment into sewers has, in the past, been strongly resisted on the grounds of risk of breakdown and the heavy cost of inspection and maintenance. In recent years, however, interest has been growing in the development of mechanically-raked screens on storm overflows of the side-weir and stilling-pond type, designed on lines similar to those used at the inlet to sewage treatment works, the screenings being raked off and dropped back into the channel conveying sewage to treatment. An inspection of a number of installations of this type, recently carried out on our behalf, suggests that, subject to careful attention to detail and to choice of equipment, such installations can produce effective results with reasonable inspection and maintenance and at reasonable cost. Where, therefore, amenity considerations are of particular importance, we suggest that the installation of mechanical screening be given careful consideration, notwithstanding the possible disadvantages.

Overflows with downstream storage

269. The occurrence of a strong "first flush" of storm sewage has been abundantly confirmed in the research work reported in Chapter 3, Discharge of a given volume of this first flush adds much more pollution

to a stream than an equal quantity of storm sewage arriving later in the storm, and also adds it at a time when the receiving stream may well offer less dilution for it. It would clearly be an improvement if the first flush could be impounded for discharge to the river at the end of the storm or, much better still, for discharge to the sewer when the storm flow had subsided. This could be achieved by a suitably placed storm tank or similar device at the overflow.

270. Ideally, the tank and overflow would be so contrived that the sewer downstream of the overflow would take up to, but no more than, the flow stipulated by the "setting". As soon as this was exceeded, all excess flow would enter the tank until it was full and the excess flow arriving after that time would discharge direct to the river. As soon as the incoming flow fell below the setting, the impounded first flush would begin to discharge to the foul sewer to make up the setting rate, until the tank was empty. The tank would be self-cleansing. How nearly the ideal could be approached in practice would depend upon circumstances. It might, for instance, be necessary to pump from the tank at the end of the storm, and it is unlikely that the pumping rate would keep exactly in step with the rate of fall in flow arriving at the overflow. Thus the tank would empty less rapidly than would be ideally desirable, with loss of some potential storage capacity for a storm following soon afterwards.

271. Provision of such a tank would ensure that a greater quantity of storm sewage would reach the treatment works and be treated there and so would be equivalent to raising the setting. This "raising of the setting" would be effected without increasing the maximum rate at which sewage arrived at the treatment works and would not therefore require the increase in sewerage capacity, pumping capacity and sedimentation capacity that would be necessitated if the same effect were being sought simply by a straightforward raising of the setting of the overflow. It would also have the advantage that no overflow would take place, however great the flow in the upstream sewer, until the cumulative increase over the actual setting exceeded the volume of the tank. In some short but heavy storms, there would be no discharge at all, whereas discharge might occur (in the absence of a tank) with even higher-than-normal settings in such cases.

272. In all storms, the commencement of discharge would be considerably delayed and this in itself would be useful because discharge would be more likely to take place to a stream already swollen by the rain. The fact that the storm sewage eventually discharged would be that which arrived later at the overflow would usually mean that, on average, it would have lower suspended-solids and BOD contents. Such a tank would seem to be particularly useful in an area where the sewage was strong.

273. In order that we might give further consideration to the provision of storage, we asked the Water Pollution Research Laboratory to carry out some calculations to see if its merits could be estimated quantitatively. This was done for an ideal arrangement at Northampton and a more practical arrangement at Brighouse, and the results are given in Chapter 3 (paras. 152–155). General comparison of a storage-type overflow with other types is discussed in Chapter 4.

274. The calculations were involved and laborious; they necessitated, in effect, re-analysing the records of flow and composition for each storm on the assumption that a storage tank was present, and then comparing the results qualitatively and quantitatively with those observed in the absence of storage. The calculations were done at a time when we were more inclined to talk about multiples of dry-weather flow than we are now, but there is inadequate justification for asking for them to be repeated. Results have been expressed in a number of different ways in order that they shall be as helpful as possible. Thus, the capacities of the tanks were originally chosen as "hours' DWF", but are also expressed in other ways in Table 17.

275. It will be seen from Table 17 that the use of a 2-hour tank at Northampton would have resulted in overflow on only 59 per cent of occasions when it would otherwise have taken place. The total duration of overflow would have been reduced to 61 per cent, and the total volume spilled to 79 per cent. The pollution overflowed would have been reduced even more than the volume (because the first flush would have been held back) to 60–65 per cent depending on the pollution index used. Increasing the capacity of the tanks would have kept more pollution out of the river, but would have given a diminishing return per unit of capacity provided.

276. Direct comparison of the results at Northampton and Brighouse, as set out in Table 17, has to be done with caution, because Brighouse was a special case with a low side-weir and an exceptionally large per-capita dry-weather flow (97 g.h.d. as against 33.6 g.h.d. at Northampton) which included a considerable amount of infiltration. It is clearly therefore not correct to compare the 2-hour and 6-hour tanks at Northampton with the 2-hour and 6-hour tanks respectively at Brighouse and expect to get mutually supporting results. It is, however, of interest to compare the 2-hour and 6-hour tanks at Northampton with the 1-hour and 2-hour tanks respectively at Brighouse, which makes some allowance—albeit a very approximate one—for the widely differing conditions. Such a study leads us to conclude that the Brighouse results add some weight to the Northampton results, but we base our conclusions mainly on the latter.

277. It is plain that the tank at the overflow does a very useful job. It is difficult, however, to answer the question—"How useful and is it worth it?"—because it cannot be compared with any other way of doing precisely the same job. We can, nevertheless, take overall figures and compare the job done by the tank at the overflow with the extra setting which would be required for equivalent annual performance. The results are given in Table 17 in terms of DWF. (In interpreting the Brighouse results the footnote must be borne in mind—the present overflow structure,

being a low side-weir, passes a greater volume to treatment than it should when the total flow exceeds the setting. The actual setting is 6.2 DWF, but in terms of volume discharged per annum it is equivalent to 9.2 DWF.)

278. At Northampton a 2-hour tank (equivalent to about three gallons per head) would ideally reduce storm sewage pollution by about 35 per cent, or by as much as raising the setting from about 6 DWF (170 gallons rainwater per head per day) to about $8\frac{1}{2}$ DWF (say 250 gallons rainwater per head per day).

279. In the light of all this, and bearing in mind the fact that the Northampton results might not apply to areas of a different character, the nearest we can get to the quantitative in a recommendation about storage tanks on overflows is to round-up the above figures. This leads us to suggest that, if an overflow is unsatisfactory and its effects need to be reduced by about a half, then instead of taking an extra 100 gallons per head per day of rainwater into the downstream sewer by raising the setting (and, of course, providing sewer and settling capacity and pumping capacity if necessary), it is worth considering the alternative of a storage tank at the overflow of approximately four gallons per head capacity. Its practicability and cost in relation to raising the setting will decide the issue.

280. In considering practicability and cost, it is worth bearing in mind, in appropriate cases, the possibility of providing storage by means of an oversized sewer as an alternative to a tank. Such a solution might commend itself when the construction of the overflow is coupled with re-sewering, and storage might be provided by inserting, downstream of the overflow, a section of over-sized sewer of such length and diameter as to give the capacity required, with suitable control at the downstream end.

281. In all cases where storage is provided, whether by tanks or oversized sewers, it is important that careful attention is given in design to ensure against cumulative deposition of sludge and grit.

CHAPTER 8. STORM TANKS

Present-day practice

282. Present-day practice for treatment of storm sewage is to pass all flows up to 3 DWF (with an adjustment when infiltration is large) to the main treatment units and to pass flows in excess of 3 DWF and up to 6 DWF to storm tanks for temporary storage and—when overflowing from the storm tanks occurs—for partial treatment by settlement. When properly operated, all storm tanks are full before any overflowing occurs, and they are emptied as soon as possible after the end of the storm and their contents passed through the main treatment units mixed with the incoming sewage flow.

283. When the maximum flow to the treatment works is 6 DWF and full treatment is given to 3 DWF, it is the common practice to provide storm tanks of 6 hours' DWF capacity which means that the minimum settlement time for the storm sewage is 2 hours. Sometimes one tank is used as a "blind" tank, which is not provided with an effluent weir, and is so arranged that it receives the initial "first flush" discharge and stores it for subsequent full treatment. The other tanks are sometimes arranged to fill in sequence, but when overflowing commences, they operate in parallel. When one tank is used to store the first flush, the settlement time available when overflowing occurs is, of course, decreased.

284. Where the flow reaching the treatment works exceeds 6 DWF, some designers arrange to pass the whole flow in excess of 3 DWF through storm tanks (sometimes enlarged), while others arrange a storm overflow direct to the stream for the flows over 6 DWF.

Field investigations

285. In order to assist us in establishing whether current practice is satisfactory, we sought the co-operation of local authorities where there were sewage treatment works considered suitable for a study of storm tanks and we now describe the main investigations, which were those carried out at the Northern Sewage Works of the Borough of Royal Tunbridge Wells and at the Blithe Valley Sewage Works of the City of Stoke-on-Trent.

286. It was appreciated that it might be difficult to record every occasion when there was flow into the storm tanks, but the objective was to obtain records for most of the occurrences over a period of about two years and this was achieved. Among the information recorded for each storm was the time when flow into the tanks started and stopped, the state of the tanks at commencement of inflow (whether empty, or, if not, the approximate contents), the time when flow out of the tanks to the stream started and stopped, and the contents of any tanks partly filled at the end of the storm. Records of flow and rainfall were also kept. A sample was taken as soon as possible after inflow started and subsequent samples taken at 15-minute intervals for the first hour and thereafter at intervals of 30 minutes or 60 minutes according to the character of the storm. Similar procedure was followed for flow out of the tanks. Each sample was analysed for BOD and suspended solids, with additional determinations at the discretion of the authority.

287. The basic details of the two sites were as follows:

(a) *Tunbridge Wells.* At the time of the investigation, the area was drained by two main sewers, one of which served a population of about 7000 on a strictly separate system and the other a population of about 16 000 served by both combined and separate-system sewers. The dry-weather flow to the treatment works was about 900 000 g.p.d. and there was a storm overflow on the combined-sewer outfall about 1000 yards upstream of the treatment works. This overflow commenced to operate at a rate of flow equivalent to about 8 times the dry-weather flow in that sewer. The setting of the overflow weir to the storm tanks was equivalent to about 3.5 DWF from the whole area and all the excess flow reaching the treatment works was passed to the storm tanks. There were two mechanically-scraped storm tanks of equal size and total capacity 6-hours design DWF, which represented about $6\frac{3}{4}$ hours' DWF capacity at the time.

The Northern Sewage Works are situated on the outskirts of the town and serve an area which has many sewers with steep gradients. The time of concentration of the system serving the "combined" area was estimated to be about 10 to 12 minutes.

(b) *Stoke-on-Trent.* The Blithe Valley Works serve an extensive area in the City of Stoke-on-Trent and in the Rural Districts of Cheadle and Stone. At the time of the investigation, the area was drained mostly by combined-system sewers with storm overflows to restrict the flow discharged to the main trunk sewers laid in the Blithe Valley and in the adjoining Tean Valley.

The population of the area draining to the treatment works at the time was about 28 000 and the dry-weather flow was about 1 m.g.d. The bulk of the flow reaching the upper part of the main trunk sewer was controlled by storm overflows designed to operate at 6 DWF, but greater flows could run off from the lower areas. During the investigations the flow reaching the sewage treatment works never exceeded 10 DWF for any appreciable period and for only about 1 per cent of the time did it exceed 6 times the dry-weather flow from the whole area. The setting of the overflow weir to the storm tanks was equivalent to just over 3 DWF.

There were three storm tanks of equal size and of total capacity 9-hours design DWF which represented about 9½ hours' DWF capacity at the time.

The main trunk sewer through the area is over 9 miles long and the time of concentration of the system was estimated to be about 5 hours.

288. The study extended over a period of rather more than 2 years at Tunbridge Wells and just under 2 years at Stoke-on-Trent. At the former site, records were obtained covering 223 occasions when flow into the storm tanks occurred; some 1500 samples of inflow and 400 samples of tank effluent were analysed. At the latter site, the records covered 86 occasions when flow into the storm tanks occurred, some 600 samples of inflow and 300 samples of tank effluent being analysed.

Strength of storm sewage and tank effluent

289. As might be expected, there were extremely wide variations in the quality of individual inflow samples. The overall averages of the BOD and suspended-solids values of the inflow to the storm tanks at Tunbridge Wells were 123 mg/l and 198 mg/l respectively; corresponding figures for Stoke-on-Trent were 151 mg/l and 374 mg/l. Closer examination of the two sets of results showed that, in general, the strength depended upon the time after commencement of the storm and the time of day, but the variation of strength with time had a different pattern at the two sites as is shown in Table 22.

TABLE 22. Variation of strength with time

Time	Tunbridge Wells		Stoke-on-Trent	
	BOD (mg/l)	Suspended solids (mg/l)	BOD (mg/l)	Suspended solids (mg/l)
At start of inflow	275	460	240	457
After 15 minutes	210	375	266	520
After 30 minutes	145	270	270	600
After 45 minutes	115	225	200	550
After 75 minutes	105	165	142	405

290. At Tunbridge Wells there was a pronounced "first flush" but it was not very different from ordinary sewage and was of fairly brief duration. After 30 minutes, for example, the strength fell by nearly a half. At Stoke-on-Trent, on the other hand, the "flush" was delayed and there was a general increase in strength over the first 30 minutes followed by a decrease. These differences were no doubt due to the locations of the two treatment works in relation to the areas they served and to the widely different times of concentration. The results suggested that a "blind tank" at Stoke-on-Trent would have been of less value than one at Tunbridge Wells.

291. There was evidence that night-time storm sewage was weaker than that during the day, but considering that at night the crude sewage is not only weaker but smaller in volume (so that it is diluted to a greater degree by storm water), the differences were quite small.

292. There were no very wide variations in the quality of the effluent from the tanks. The overall averages at Tunbridge Wells for BOD and suspended-solids content were 55 mg/l and 88 mg/l respectively; corresponding figures for Stoke-on-Trent were 52 mg/l and 129 mg/l. Tables 23 and 24 show how the samples of tank effluent could be classified in terms of BOD and suspended-solids content.

TABLE 23. Percentage of samples of tank effluent in various ranges—BOD

Range (mg/l)	Percentage of samples in range	
	Tunbridge Wells	Stoke-on-Trent
0–50	43	57
50–100	44	35
Over 100	13	8

TABLE 24. Percentage of samples of tank effluent in various ranges—suspended solids

Range (mg/l)	Percentage of samples in range	
	Tunbridge Wells	Stoke-on-Trent
0–100	63	64
100–150	32	20
Over 150	5	16

293. It is of interest to compare the strength of storm sewage entering the tanks with that of the settled storm sewage leaving the tanks at the same time. This illustrates the difference between, on the one hand, filling the tanks and then passing subsequent flows through them and, on the other, filling the tanks and then by-passing them direct to the river. The results for each year were examined separately and the average figures for the storms studied are shown in Table 25.

TABLE 25. Comparison of storm sewage entering tanks with settled storm sewage leaving tanks at the same time

Site		BOD in (mg/l)	BOD out (mg/l)	Suspended solids in (mg/l)	Suspended solids out (mg/l)
Tunbridge Wells	1st Year	73	47	114	84
	2nd Year	89	73	112	83
Stoke-on-Trent	1st Year	79	50	274	109
	2nd Year	96	57	175	108

294. The results show that generally it was better to use the tanks as continuous-flow settlement tanks once they were full, but there were occasions at both sites—usually towards the end of night storms—when the storm sewage flowing into the tanks was weaker than the settled storm sewage it was displacing over the weirs. It would not normally be practicable, however, to arrange for the tanks to operate in such a way as to take advantage of these occurrences.

Purification in the storm tanks

295. An assessment of purification effected can be made using those analyses which relate to storms when the tanks were empty at the start and subsequently overflowed. Table 26 gives information about this. It relates to Tunbridge Wells and shows values for BOD and suspended solids which are the averages of the first hour of inflow and the first hour of tank effluent for the relevant storms. The results for the two years are shown separately. (The results obtained at Stoke-on-Trent were not sufficient for preparation of a comparable analysis.)

TABLE 26. Purification in storm tanks—tanks empty at start and subsequently overflowing (Tunbridge Wells)

Average of first hour of flow	1st Year	2nd Year
BOD in (mg/l)	224	232
BOD out (mg/l)	58	90
BOD reduction	74%	61%
Suspended solids in (mg/l)	435	329
Suspended solids out (mg/l)	93	83
Suspended solids reduction	78%	75%

296. The reduction is suspended-solids content is about what would be expected in settlement tanks, but the reduction in BOD is a good deal greater than is normal for such treatment. This may be due to the mixing within the tank of the first hour's storm sewage with subsequent and weaker inflow. The rate of flow through the tanks must also influence the amount of purification effected and it is perhaps significant that the rate of flow to the storm tanks seldom reached 3 DWF and was most commonly in the range 1–2 DWF.

Storage

297. Out of 223 occasions when there was flow into the Tunbridge Wells tanks, there were only 50 occasions when the tanks overflowed and at Stoke-on-Trent, the corresponding figures were 86 and 39. However, not all the tank capacity at Stoke-on-Trent was used at all times because, as part of the experimental procedure, either one or two tanks were put out of action during some of the storms and so the figure of 39 must be treated with reserve. The corresponding periods of inflow and discharge were 670 hours and 210 hours for Tunbridge Wells (a reduction in period of discharge of 69 per cent) and 400 hours and 210 hours for Stoke-on-Trent (a reduction in period of discharge of 47 per cent, a figure again influenced by the way the tanks were operated).

298. The volume discharged into and out of the Stoke-on-Trent tanks with each storm was examined and related to the corresponding number of tanks in use. It was thus possible to assess the volume which would have been discharged to the stream had the installation consisted of 2, 3 or (a hypothetical) 4 tanks (approximately 6, 9 and 12 hours' DWF capacity respectively), as shown in Table 27. It will be noted that a much greater amount of storm sewage was discharged to the tanks during the first year which was the wetter of the two. (About 50 per cent more rain fell in the first period than in the second.)

299. All the foregoing figures indicate that, at Stoke-on-Trent, about half the storm sewage would have been stored in tanks with 6–9 hours' DWF capacity, and this storm sewage would ultimately have received full treatment. Furthermore, it will be seen that, in the second year, a 6-hour installation would have done about as well as would a 9-hour installation in the first year. On the basis of quantities alone, it is evident that successive increments of storage capacity gave progressively less return because, of course, each tank would have been filled less frequently than the one before.

Reduction of pollution

300. It is reasonable to assume that most of the settlement occurs in the 6-hour tanks and that subsequent tanks provide mainly storage. It is also reasonable (and not out of line with the observed behaviour) to assume that at least 50 per cent reduction in BOD would be achieved by this settlement, with little further reduction thereafter. If, therefore, we couple an assumed 50 per cent reduction of BOD with the different volumes that would have been discharged from installations of 6, 9 or 12 hours' storage capacity as shown by Table 27, we can compare roughly the polluting load that would have been discharged as effluent from the storm tanks with that which would have been discharged in the absence of storm tanks. This is done in Table 28.

TABLE 27. Effect on volume of effluent of varying the storage capacity (Stoke-on-Trent)

	Mil gal			Percentage of inflow discharged to stream		Overall percentage of inflow discharged to stream
	1st year	2nd year	Total	1st year	2nd year	
Storm sewage into tanks	17.8	8.0	25.8	—	—	—
Effluent discharged from 6-hour installation	11.9	4.1	16.0	67	51	62
Effluent discharged (actual)	11.1	3.3	14.4	62	41	56
Effluent discharged from 9-hour installation	10.0	3.1	13.1	56	39	51
Effluent discharged from 12-hour installation	8.0	2.4	10.4	45	30	40

TABLE 28. Estimated reduction of polluting load discharged

Installation	Percentage of total polluting load of storm sewage discharged as tank effluent	Percentage reduction in polluting load of storm sewage by use of storm tanks
6-hour tanks	31	69
9-hour tanks	25.5	74.5
12-hour tanks	20	80

Design considerations

301. In our Interim Report, we said that, on the evidence then available to us, the storm-tank capacity normally provided at treatment works—6 hours' DWF capacity for flows between 3 and 6 DWF (2 hours' retention at maximum flow)—appeared to be about the optimum. Further consideration leads us to modify this opinion and, moreover, we have now suggested that the "norm" for setting storm overflows be expressed in a different form which will, in many cases, result in greater flows reaching the treatment works. It therefore becomes necessary to see what effect this will have upon storm tanks of conventional capacity.

302. Before considering what flow will reach the storm tanks, it is first necessary to determine the flow that should be passed to full treatment. It is well known that the peak daily rate of flow in a sewer in dry weather can rise above 2 DWF (at Brighouse it rose to about 2.5 DWF) and we think that the principal justification for the normal practice of designing full-treatment units on the basis of a maximum rate of flow of 3 DWF is that it ensures that there will be no by-passing of the full-treatment units in dry weather. We regard this as sound practice and since the considerations applying to storm flows do not apply to peak dry-weather flows, we have no reason to recommend a variation of the basis upon which flow to treatment has hitherto been calculated. We therefore think that the maximum rate of flow to full treatment should remain as 3 DWF, despite our suggestion of a departure from the DWF-multiple approach to storm-overflow settings.

303. As previously noted, the term DWF is variously interpreted and a definition of 3 DWF is required for use in the present context. Domestic sewage flow and flow of industrial effluent are subject to diurnal variations but infiltration is not and we therefore see no reason why any multiplier should be applied to the infiltration element. We accordingly suggest that, in the design of sewage-treatment units at treatment works where there are storm tanks, the term 3 DWF should be taken as meaning

$$3PG+I+3E \text{ gallons per day}$$

where the terms have the same meaning as in Chapter 6 (paras. 206, 214, 216), but applied to the whole area draining to the treatment works.

304. We are aware that there is a body of opinion that favours varying the multiplier for *PG* according to the size of the drainage area. It seems reasonable to take some account, in this way, of the difference in the diurnal variation of dry-weather flow in large and small drainage areas, but we have not studied this point and make no quantitative recommendation. We think, however, that it would be reasonable to reduce the industrial-effluent multiplier in cases where there was known to be balancing of industrial effluent at source.

305. Referring now to our field studies, they demonstrated the dual function of storm tanks. The tanks impounded the storm sewage so that the quantity ultimately given full treatment was increased. Had the tanks been larger, they would, of course, have impounded more storm sewage for ultimate treatment, but each additional tank would have been of less value than the one before because it would have been filled less frequently. They also acted as continuous-flow settlement tanks and brought about an improvement in the quality of the storm sewage before it was discharged as tank effluent. The settlement that was taking place was, on average, about what would have been expected with the capacity provided, and had the tanks been larger, we do not think that the quality of the effluent would have been much improved. In any case, the average settlement time would often have been longer than the nominal period because only a small percentage of storms resulted in the flow in the sewers at the treatment works reaching 6 DWF and, even then, the initial inflow would have been at a lower rate.

306. Each case needs to be determined on merit, having regard to the size and use of the receiving stream, but we have no evidence from our field studies of storm tanks to lead us to recommend that the normal provision to deal with flows between 3 and 6 DWF should be increased, nor do we think that it would have been a profitable exercise to try to find out just how big an installation would have to be to provide the right amount of storage and settlement and no more.

307. Our general view is, if anything, strengthened when we consider the situation in the country as a whole. We have already shown that many existing overflows are of the low side-weir type, which passes flows to the treatment works in excess of the nominal setting, and we think it reasonable to infer that at many works the maximum rate of flow to the storm tanks is, on occasions, in excess of the design figure. At the same time, we are not aware of any general complaint from river authorities that storm tanks designed on present-day practice are grossly unsatisfactory in operation.

308. We conclude therefore that the capacity normally provided is on the whole adequate and may be even generous, although we are conscious that tank capacity is comparatively cheap to provide and operate, and that only minor financial savings would be achieved by providing tanks of smaller capacity than normal.

309. We now have to examine what would be the effect on storm tanks of our suggested new "norm" for overflow settings. In this consideration, we are concerned only with the flow from "combined" or

"partially-separate" areas. The flow from areas on the separate system would, we have assumed, be at a maximum rate of 3 DWF from these areas (para. 219) and the whole of this flow would receive full treatment so that it can be ignored in considering storm-tank design.

310. The maximum rate of flow to the treatment works from the "combined" and/or "partially-separate" areas would be (from *Formula A*, para. 216)

$$DWF+2E+300P \text{ g.p.d.},$$

and this can be re-written as

$$PG+I+3E+300P \text{ g.p.d.}$$

The corresponding rate of flow to full treatment, according to our suggested interpretation of 3 DWF, would be

$$3PG+I+3E \text{ g.p.d.}$$

The maximum rate of flow to the storm tanks would therefore be

$$(300P-2PG) \text{ g.p.d. or } P(300-2G) \text{ g.p.d.}$$

and it is immediately apparent that, on this basis, the maximum rate of flow to the storm tanks would be independent of the infiltration and flow of industrial effluent and would tend to decrease as the domestic water consumption, G, increases.

311. The nominal retention period in a traditional storm-tank installation under these conditions of maximum rate of flow is illustrated in Table 29. Column (a) represents the case where there is domestic sewage only. This would not arise very often, because the dry-weather flow would nearly always contain elements of either infiltration or industrial effluent or, more usually, both. Columns (b) and (c) illustrate situations where these factors are taken into account. They are not necessarily typical, but merely illustrate how the nominal retention period at maximum inflow would vary when the dry-weather flow used as the basis for normal storm-tank design included infiltration and industrial effluent.

TABLE 29. Retention period (hours) at maximum flow in storm tanks of 6 hours' DWF capacity

Domestic water consumption (g.h.d.)	Maximum rate of flow through tanks (g.p.d.)	(a) Domestic sewage only	(b) $(E+I)$ equal to 50% of domestic sewage	(c) $(E+I)$ equal to 20 g.h.d.
30	240P	0.75	1.13	1.25
40	220P	1.10	1.65	1.65
45	210P	1.28	1.93	1.86
50	200P	1.50	2.25	2.10
60	180P	2.00	3.00	2.67

312. It will be seen from the table that, with the infiltration and industrial-effluent figures assumed, the nominal retention period in traditionally-designed tanks, under the conditions created by the new "norm" for overflow settings, would approach 2 hours when the domestic water consumption was between 45 and 50 g.h.d. We are therefore of the view that, whilst there is a need to improve those sewerage systems which are of a standard well below the "norm" we have suggested and which, as a result, produce unsatisfactory conditions, there is no pressing need for an accompanying increase in the capacity of traditionally-designed storm tanks to allow for the fact that the raising of the overflow settings will result in larger flows reaching the treatment works.

313. It does not follow, however, that the traditional basis of design, expressed in terms of sewage DWF, is the right or most logical basis to use in the future. We have shown that, under the new arrangements we propose, the flow to the storm tanks would depend on the domestic sewage flow only and, as emphasised by Table 29, widely different retention periods result from the traditional design approach. We think that there is a clear case for defining storm-tank capacity simply in "gallons per head" in the same way, in fact, as we have defined the surface water retained in the sewer downstream of an overflow.

314. Table 30 shows how the nominal retention periods would vary under different conditions if the tanks were designed on this basis, assuming that our recommendations for the basis of design of storm overflows and flow to treatment had been followed.

TABLE 30. Retention periods (hours) at maximum flow in storm tanks designed on the basis of "gallons per head"

Domestic water consumption (g.h.d.)	Tank capacity (gallons per head) 8	10	12	15	20
30	0.80	1.00	1.20	1.50	2.00
40	0.88	1.10	1.32	1.65	2.20
50	0.96	1.20	1.44	1.80	2.40
60	1.07	1.33	1.60	2.00	2.66

315. For any particular design capacity, there is not a very big difference in the retention period as the water consumption varies, and it would appear permissible to select a design figure, or a range of figures to suit the average case.

316. We have said that, in our view, 2 hours' retention at maximum flow is adequate and perhaps even generous, and with this in mind it is clear from Table 30 that a design basis of 20 gallons per head would be unnecessarily high. It also appears that 10 gallons per head would be too low, and we think that it would be reasonable to adopt, for normal conditions, a design figure of 15 gallons per head. This would give retention periods within the range we consider to be about right.

317. It might be argued that this means virtually doubling the capacity of traditionally-designed tanks in cases where the domestic water consumption is low, and that this would be excessive, bearing in mind the improvements that are likely to be brought about by the adoption of the new and higher settings. Comparison of Tables 29 and 30, however, shows that, in the lower ranges, the retention provided by the new design method would generally not be far out of line with

what would be provided in "6-hour" tanks when there is a reasonable amount of infiltration and industrial effluent. In the higher ranges, the retention provided by the new method would be less.

Storm tank operation

318. We think it would be useful to set down a few points (some of which may be somewhat elementary, yet all of which we consider to be important) concerning the operation of storm tanks.

319. It is usually better to arrange for tanks to fill in sequence but to overflow simultaneously when full. One full tank is better than two half-full tanks, especially if they are of the flat-bottomed type, and have to be manually cleaned.

320. Our field studies have not demonstrated that a "blind" tank will always be an advantage, but the evidence suggests that such a tank would be of more benefit with a drainage system having a short time of concentration. A study on the lines we carried out would show whether, in any particular case, the strength of the "first flush" merited the use of such a tank.

321. The importance of emptying the tanks and removing the sludge as soon as possible after the end of the storm should be emphasised. This is an elementary point, yet is one that is not always given the attention it requires.

322. When we approached the river boards about their views on storm overflows (as we have described in Chapter 2), a number of the boards were of the opinion that land irrigation of storm-tank effluent would be beneficial. We have not covered this aspect in our field work, but where there is a suitable area of land, not required for some other more important purpose, and which could be adapted at low cost, we see no reason why the storm-tank effluent should not be given some treatment. We make no recommendation, however, that land treatment be introduced as part of the normal process of storm-sewage treatment.

323. At some sewage treatment works the practice is being adopted of combining the use of storm-tank capacity with that provided for sedimentation (either primary or secondary), so adding to the sedimentation capacity available for use in dry weather. When this practice is adopted, the advantage of storing storm sewage for subsequent full treatment is usually lost, but there may be compensating advantages in some cases, such as the elimination of separating weirs and the possibility of passing all the sewage arriving at the works through the filters. We have not studied in detail the possibilities of the many variations of this practice, but we do not think that any of them would justify an equivalent "storm tank capacity" smaller than we have recommended for the conventional case.

CHAPTER 9. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

324. We submitted our Interim Report in 1963 with considerable diffidence. Our investigations were incomplete, but we believed that, because practical decisions have continually to be made and cannot be easily altered for a number of years, it was better that they be made with inadequate guidance than with none at all.

325. Since the Interim Report was published, we have completed the special studies listed in Chapter 1 and this has enhanced our knowledge on some aspects of the problem. Nevertheless, these studies have shown that it will be a long time before some of the more important questions can be answered. The subject is a complex one and, although there is much evidence available on certain aspects, we are conscious of the fact that some of the advice we give is based on collective opinion rather than on established fact.

326. We do not think our final report is inconsistent with the Interim Report, but naturally it supersedes it.

327. It is appropriate at this point to review briefly the ground we have covered in the Report and to pick out some of the more important points that have emerged.

328. We began with a brief historical review of the events which led to the need for storm overflows and to the development of present-day practice, and we described how difficulties had arisen, leading to the appointment of the Committee.

329. In Chapter 2, we discussed the question of completely separating all surface water from existing combined and partially-separate sewerage systems, and from the information which we had been able to assemble, we concluded that, however desirable it might be, the complete separation of surface water from existing combined and partially-separate sewers would not be economically practicable. We suggested that the best that could be achieved would be separation of surface water as opportunities occurred, for example, when re-development schemes were carried out.

330. We went on to describe our survey of existing storm overflows which was an attempt to assess the magnitude of the problem as seen by the river boards, and this produced valuable information about the types of overflow in use and the extent to which they were, in the opinion of the river boards, satisfactory or otherwise. We felt that this information was particularly useful because, in the end, the river boards (now river authorities) have the duty to control storm-overflow discharges in their areas. This survey showed that there were special problems of closely-spaced overflows in some of the bigger towns but, apart from these, more than 80 per cent of storm overflows having the traditional setting of 6 DWF or higher were considered satisfactory. It was also evident that a worthwhile improvement would result if the discharge of gross solids were better controlled.

331. In Chapter 3, we described our field investigations on the flow and composition of storm sewage. The first part of the chapter was concerned with examination of flow variations in dry and wet weather at three sites, and showed how the duration and frequency of operation of an overflow, and the volume discharged, were related to overflow setting. Reasonably consistent results indicated the possibility of predicting the duration and volume of discharges from other overflows in comparable areas. Next, the factors which influence the strength of storm sewage were examined, and an assessment was made of the polluting load discharged and of how this might vary with overflow setting. Finally, we considered the provision of storage tanks at storm overflows, and showed how the benefit from this could be equivalent to that from a substantial increase in the setting.

332. Chapters 4 and 5 dealt with laboratory-scale and field-scale experiments on models of different types of storm overflow. Both series of experiments provided information which enabled us to compare the performance of some widely-used types. We found that, once the screenable material had been removed, there was little difference in the composition of the sewage flow passed to treatment and the storm-sewage flow, thus indicating that the impurities were not amenable to separation by hydraulic devices. The experiments suggested that the provision of storage at or downstream of the overflow was the most effective way of reducing total polluting load discharged as storm sewage, for any fixed setting.

333. Chapter 6 dealt with the setting of storm overflows, and we began by pointing out the shortcomings of the present practice of defining the setting in terms of a multiple of the dry-weather flow. We concluded that this practice would have to be changed. It seemed to us that the right way of expressing the setting would be in the form of the sum rather than the product of two terms. One of these terms would be the dry-weather flow and the other would be a term defining the amount of run-off to be retained in the sewer before overflowing started.

334. We then looked at our survey of existing overflows and the opinions of the river boards on these overflows, and we came to the conclusion that there was a case for a modest improvement on the traditional practice, but no justification, in the general case, for a radical improvement. This led to our suggesting that, taking account of likely increases in future water consumption, a desirable and modest improvement would be achieved by permitting a surface-water run-off equivalent to 300 g.h.d. to be retained in the sewer before discharge of storm sewage commenced, with an additional allowance to provide dilution for industrial effluent in the sewage.

335. The formula thus derived had the merit of simplicity, but we were conscious of the fact that

it was not scientifically derived and did not take account of many of the factors that influence storm-sewage discharge and its effects. We therefore turned to the experimental work described in Chapter 3. There were some formulae developed from that work which, we thought, might lead to better account being taken of the various relevant factors.

336. We discussed these formulae at great length and concluded that, at the present time, we could not justify their use in preference to the more simple formula based mainly upon the evidence of the survey. We thought, however, that the merits of the formulae developed in Chapter 3 ought to be tested by applying them to cases where the simpler formula was to be applied.

337. In Chapter 7, we discussed the design of overflows and gave our views on some of the more important considerations such as flow control and control of the discharge of gross solids. We also discussed the provision of storage based upon the evidence of the experimental work described in Chapter 3.

338. Chapter 8 dealt with storm tanks and we first described field investigations carried out on storm tanks at two sites. These studies, together with other evidence, led us to conclude that the capacity provided by the traditional design was generally adequate, and that there was no pressing need for improvements in overflow settings to be accompanied by increases in storm-tank capacity. We then went on to show that the traditional design basis, despite the adequate results it had provided in the past, was not, however compatible with our recommendation for overflow settings, and we therefore suggested a new design approach for use in the future.

Conclusions and recommendations

339. As a result of the investigations which have been made at our request, and the information we have gathered from various sources, we know a great deal more about storm overflows and their properties, behaviour and effects than we knew before. Even now, however, knowledge on some aspects of the subject is still scanty, for reasons which have been explained. Given the necessary facilities, staff and resources many of the gaps in our knowledge could eventually be filled, but it would take a long time to do so.

340. We would have liked the support of more established facts for most of our conclusions, and we realise that some of them depend more than they should on assumptions and opinions which, though they seem reasonable, could eventually be proved inaccurate to a greater or lesser extent. This is the reason why our recommendations are not as precise as many would wish them to be, and why we hope that research to obtain further information will in due course be carried out.

341. Storm overflows are only necessary when the sewerage system is combined or partially-separate. We estimate that, at the time of the I.S.P. survey (Table 1), there were some 36 million people in England and Wales (about 76 per cent of the total population) living in areas so sewered. (Paras. 30–32.)

342. In recent years there has been a marked tendency to adopt the separate system for new developments. We do not disapprove of this trend (whether or not it can be demonstrated that it is always worth the cost) and we think that it would be generally undesirable to adopt a policy which resulted in the construction of new sewerage systems with storm overflows. In pursuing a policy of separate-system sewerage, however, it should be borne in mind that there will be areas from which the flow of surface water is likely to be of a highly polluting character because of the activities (largely industrial) carried on in those areas. It would clearly be wise in those cases to discharge such surface water to the foul-sewerage system, and appropriate allowance should be made in the design of the system for the surface water to be taken into it. (Paras. 33–38.)

343. It would be unrealistic to contemplate eliminating, over the next few decades, all storm overflows, either by enlarging the sewer capacity or by providing separate sewers for the surface water. The cost of any such project would be prohibitive. However, the opportunity might be taken in re-development schemes to separate surface water in whole or in part. (Paras. 39–42.)

344. Our survey indicated that there are between 10 000 and 12 000 storm overflows in England and Wales and that some 37 per cent of these are unsatisfactory either in themselves or in association with others. Of those which were reported to be set at 6 DWF or higher and which could be assessed individually by their observable effect, only about 18 per cent were classed as unsatisfactory and this figure would have been reduced to about 14 per cent had efficient means of retaining gross solids been provided. Such figures do not establish a general need for any radical improvement in the normal overflow settings; in fact they indicate that much of the trouble arises from those overflows set lower than the hitherto traditional setting of 6 DWF. (Paras. 50–58.)

345. We think that there are generally too many storm overflows and that sewerage authorities could, with advantage, examine their systems with a view to using overflows and sewer capacity to the optimum extent. We believe that there should be close co-operation between sewerage authorities and river authorities in the study of this aspect and in the planning of remedial measures. (Paras. 58–60.)

346. We consider that the custom of expressing the setting as a multiple of the dry-weather flow is basically unsatisfactory. It is better to use the sum (rather than the product) of two terms, one being the dry-weather flow and the other the amount of surface water to be retained in the sewer before overflow commences. (Paras. 204–205.)

347. The results of the experimental work described in Chapter 3 make it possible, when combined with assumptions concerned with typicality, aims and some factors which have not been specifically studied, to derive formulae from which figures for the second of

the two terms can be calculated. We would like to see some experience in the use of these formulae with some comparative studies of the behaviour of overflows so set, in order to ascertain whether they adequately take into account the main factors affecting storm-sewage flows and their effect on receiving streams. Such studies would certainly take several years; meanwhile we cannot recommend such formulae for general adoption. (Paras. 223–245, 249–254; *Formulae B* and *C*.)

348. The formula we recommend for the normal case is an empirical one, but it is more readily comparable with the traditional one and is easily seen to be an improvement upon it in form. Where the existing dry-weather flow (sewage and infiltration) is high, the new formula may give results very little different from the old. Where the existing dry-weather flow is low, the new formula will mean an increase in the setting. On average, it corresponds with what we have called a modest improvement, and this is what we think the situation calls for. The formula we recommend is:

Setting $(Q) = DWF + 300P + 2E$ gallons per day, *Formula A*, where

DWF is the dry-weather flow in gallons per day (the average daily rate in dry weather including infiltration water and industrial effluents) of the "combined" and/or "partially-separate" areas draining to the point of overflow,

P is the population of these areas, and

E is the volume of industrial effluent, in gallons, discharged to the sewer from these areas in 24 hours.

Where areas drained on the separate system discharge sewage to the combined and/or partially-separate system upstream of an overflow, the quantity passed to treatment as defined by the above formula should be increased by an amount equal to 3 DWF from the "separate" areas. (Paras. 206–219, 248–254, *Formula A*.)

349. We consider the formula suitable unless there are special circumstances, such as overflows into very small (or very large) streams, which might justify an increase (or reduction) in the figure of 300, or where there may be abnormal discharges of industrial effluent, calling for a variation in the term "$2E$", or where there is reason to believe that a bare allowance of 3 DWF from any connected "separate" area would be inappropriate. Any such departure from the normal value would need to be justified on the merits of the case. We would point out that the figure of 300 g.h.d., though, in our view, reasonably based, is a rounded-off figure and so we do not think that it would be possible to justify variations in it of less than, say, 50 in magnitude. (Paras. 220 and 221.)

350. Our survey showed that a very high proportion of individual overflows set according to previous standards were satisfactory, and there is no obvious reason why these should be altered. However, as the overflows come up for reappraisal, we hope the settings will be made to conform with what we recommend. (Para. 255.)

351. Of the overflow structures studied, the high side-weir and stilling pond preferentially retain gross solids in the sewer but not to any really worthwhile extent, and performance depends very much on rate of flow. With downstream control, they give accurate control of flow to treatment, which is highly desirable. The choice between these two types would sometimes be determined by practical site considerations. (Paras. 171–173, 175, 179, 188–193, 195, 197–203.)

352. The particular vortex overflows we studied have no advantage over the high side-weir and stilling pond, but we recognise that the geometry of the model vortex overflows might have been improved. (Paras. 171–173, 176, 188–193, 196, 198–203.)

353. The low side-weir is inefficient, particularly in respect of flow control, and we have no evidence to support its continued use. It is so inefficient in providing proper separation of flows that we think an early opportunity should be taken of improving existing overflows of this type. Merely to build up the sills so that they become high side-weirs could, in many cases, cause difficulties lower down the system. However, if coupled with downstream control, conversion to high side-weirs would be relatively cheap and altogether desirable, provided that it did not cause unacceptable conditions of surcharge upstream. (Paras. 171–174, 188–191, 194, 197, 200–203, 257, 258.)

354. If the discharge from a storm overflow can be delayed by providing storage capacity downstream of it, much of the very strong first flush can be retained in the sewer for ultimate treatment, without increasing the capacity of the downstream sewers or the treatment plant. Such storage capacity could take the form of separate storage tanks or an over-sized (tank) sewer between the orifice (or similar control) and the overflow. When conditions permit, we recommend that consideration be given to some such installation. We also think that where storage is provided, there is justification for modifying the normal overflow setting and although, with our present knowledge, we cannot be precise, it would be reasonable to suggest, as a rough guide, that the provision of a storage tank (or equivalent) of about 2 hours' DWF capacity at the overflow would justify reducing the figure of 300 in the recommended formula to about 200. (Paras. 152–155, 171–173, 177, 179, 201, 202, 269–281.)

355. Storm overflows (with the possible exception of the low side-weir) can generally be expected to perform more efficiently when the upstream sewer is laid at a sub-critical gradient. (Paras. 171–179.)

356. We consider that, wherever practicable, some form of hydraulic control should be incorporated as a part of all storm-overflow installations. This could take the form of an orifice or a length of throttle pipe that would control with some degree of accuracy the maximum rate of flow passed forward. The orifice throttle should be of such a size as to be free from danger of blockage. (Paras. 263–266.)

357. Although gross solids constitute only a small proportion of the polluting load, they are aesthetically objectionable and the practice of introducing scum-

boards to retain them is, at best, only partially successful. Where amenity considerations are of particular importance, we recommend that consideration be given to the use of purpose-made mechanically-raked screens. If hand-raked screens are installed, frequent inspections and maintenance are of the utmost importance. (Paras. 267 and 268.)

358. Our studies have led us to conclude that the storm-tank capacity normally provided in association with the traditional 6–DWF overflow setting is adequate and perhaps more than adequate. (Paras. 305–308.)

359. The adoption of our recommended new "norm" for overflow settings is likely to result, in many cases, in higher peak flows reaching the sewage treatment works with, in consequence, higher flows being passed through existing storm tanks. This could require additional storm-tank capacity, but we do not think it should be assumed automatically that this will be so. A reasonable case should be made before it is demanded. (Paras. 309–312.)

360. It is our view that the quantity of sewage given full treatment should remain, as at present, three times dry-weather flow as defined in paragraph 303. This means that, where our recommended formula for overflow settings is adopted, the quantity separated for partial treatment in storm tanks will not be a multiple of dry-weather flow as before. It will be dependent on the domestic sewage flow and will actually tend to get smaller as the domestic sewage flow increases (see para. 310). It is therefore clear that the traditional method of designing storm-tank capacity on the basis of "hours' DWF" is not compatible with what we recommend. We think it should be defined in "gallons per head" in the same way as we have defined the amount of surface water that will be carried to treatment downstream of an overflow. (Para. 313.)

361. For new works and for existing works where an increase in storm-tank capacity has been proved necessary, we recommend that capacity be provided equal to 15 gallons per head of the population of the "combined" and/or "partially-separate" areas draining to the treatment works. (Paras. 314–317.)

Future investigations

362. The research and studies we have instigated have assisted us in reaching our conclusions but have at the same time made us conscious of the gaps in our present knowledge.

363. We consider that there is an urgent need for a study to be made of the effect of intermittent discharges of storm sewage on streams. This is clearly within the province of the Water Pollution Research Laboratory and we would have asked them to do it early in our investigations, had it not been for difficulties which we have already described. In view of the importance of the subject, we think renewed attempts should be made to overcome these difficulties. The first task would be to select a suitable site, and no doubt river authorities could be relied upon to make known to the Laboratory, locations which might prove useful.

364. We also believe that information could be collected and further knowledge acquired on the subjects of storage and the control of the discharge of gross solids by the study of actual overflows. This would require the introduction of additional facilities (for sampling, metering, etc.) at suitable existing overflows, or when new ones were constructed. We think that this work would be best done under the aegis of the Ministry of Housing and Local Government, who, with their knowledge of schemes involving construction of storm overflows, should be able, with the co-operation of local authorities, to select the most suitable sites, arrange for the necessary additional works, and co-ordinate the subsequent studies.

365. Similarly, the Ministry could, we think, take the initiative in arranging for information on storm-tank operation, which is obtained already by some authorities, to be collated in a form which would facilitate statistical analysis and direct comparison. This would in all probability bring to light generalisations on storm-sewage composition and storm-tank performance which could ultimately provide a sound basis for improved practice.

366. We do not recommend that special research be undertaken on the design of storm tanks. In so far as their purpose is storage, design details are irrelevant; in so far as their purpose is sedimentation, the design requirements are little different from those of ordinary sedimentation tanks and do not justify separate investigation.

367. It will be several years before the results of the above work are available and the practical effects of our recommendations can be observed on any reasonably large scale, and so we do not think that there would be any merit in reviewing the situation within, say, the next five or seven years. At some stage after that, however, it could well be that the situation would merit re-consideration by a representative committee.

Acknowledgments

368. In our work we have been privileged to receive most willing co-operation from many local authorities, from several river authorities (and their predecessors), from research laboratories and from individuals. Advice and assistance on a wide range of subjects has been readily given, and we wish to express our thanks for it. It is impossible to mention all of the above by name, but we are particularly grateful to the Corporations and officers of the County Boroughs or Boroughs of Bradford, Northampton, Brighouse, Luton, Stoke-on-Trent and Royal Tunbridge Wells, who co-operated to the full in the extended field studies described in Chapters 3, 5 and 8.

369. The field work at Northampton, Brighouse and Bradford was carried out for us by the Water Pollution Research Laboratory. Each investigation occupied more than two years and was fraught with difficulties

which would have daunted many. We are glad to make special mention of the staff concerned for their zeal, determination and skill, both in the carrying out of the work and in the analysis of the enormous amount of data obtained at the three sites. Much of the detailed work at Luton was carried out by the same Laboratory, but the important contribution of the Hydraulics Research Station is warmly acknowledged, as also is their assistance in the other experimental studies.

370. We have had seven Secretaries and would like to thank them all. But the chief burden has fallen upon our three Technical Secretaries, all of whom have served us well, but among whom we would select for special mention Mr. A. J. Herlihy, for to him has fallen the task of drafting almost all our report, revising it extensively (for we feel we have been hard to please) and editing it. In all this he has shown great competence, sympathetic understanding and never-ending patience. To him our best thanks are due and are gladly tendered.

371. Also assisting in the preparation of the report was Mr. R. N. Davidson of the Water Pollution Research Laboratory. He was specially responsible for drafting Chapter 3, and has served on our Drafting Committee throughout.

We have the honour to be, Sir,

Your obedient servants,

(*Signed*) R. A. Elliott (*Chairman*)

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Mrs. J. Ash	A. J. Herlihy
Secretary	*Technical Secretary*

April 1969

APPENDIX 1. TERMS AND DEFINITIONS

Sewage	The contents of sewers carrying the water-borne wastes of a community.
Foul sewage	A term used to distinguish between sewage (as defined above) and the contents of sewers carrying surface water only.
Industrial effluent	The water-borne wastes of industry.
Infiltration	The unintended ingress of ground water into a drainage system.
Subsoil water (Ground water)	Water occurring naturally in the subsoil.
Dry-weather flow	(1) The sewage, together with infiltration, if any, flowing in a sewer in dry weather; (2) The rate of flow of sewage, together with infiltration, if any, in a sewer in dry weather.
Surface water (Storm water)	Natural water from the ground surface, paved areas and roofs.
Storm sewage	Foul sewage mixed with relatively large quantities of surface water.
Combined system	A drainage system in which foul sewage and surface water are conveyed by the same pipes.
Separate system	A drainage system in which foul sewage and surface water are conveyed in separate pipes.
Partially-separate system	A modification of the separate system, in which part of the surface water is conveyed by the foul sewers.
Dual-purpose sewer	A sewer which conveys both foul sewage and surface water.
Storm overflow	A device, on a combined or partially-separate sewerage system, introduced for the purpose of relieving the system of flows in excess of a selected rate, the excess flow being discharged to a convenient watercourse.
Rainfall intensity	The amount of precipitation occurring in a unit of time.
Run-off	The discharge of water derived from precipitation on a surface.
Impermeable area	In the context of this Report, the total of the roofed and paved areas directly connected to the sewerage system.
Time of concentration	The longest time taken for the rain falling on the drainage area to reach the point under consideration (generally, in this Report, the storm overflow).
Types of Storm Overflow	
Low side-weir	A weir constructed along the length of the sewer, with the crest of the weir below the level of the horizontal diameter of the upstream pipe.
High side-weir	A weir constructed along the length of the sewer, with the crest of the weir above the level of the horizontal diameter of the upstream pipe (preferably near the soffit).
Stilling pond	A chamber designed with the object of reducing the amount of turbulence in the vicinity of the overflow. The overflow arrangement may consist of a weir or weirs at the sides or end of the chamber; alternatively, siphons may be used to discharge the excess storm sewage.
Siphon	An overflow where siphon pipes or ducts are used instead of a weir to discharge the excess storm sewage.
Vortex with central weir	An overflow in the form of a circular chamber with a peripheral sewage channel, the inner wall of which is constructed as a circular weir for discharge of excess storm sewage through a central shaft.
Storage-type	An overflow incorporating specially-provided storage capacity between the overflow weir and the downstream sewer, with the object of storing the initial flow of storm sewage and so preventing its discharge over the weir.
Leaping weir	An overflow where the dry-weather sewage is discharged to the downstream foul sewer through an opening in the floor of the chamber. Storm flows "leap" across the opening and are discharged by another outlet.
Sub-critical flow	The flow condition in which the Froude number is less than unity, and surface disturbances will travel upstream.

Super-critical flow	The flow condition in which the Froude number is greater than unity and surface disturbances will not travel upstream.
Froude number	The non-dimensional number obtained by dividing the mean velocity by the square root of the product of the mean depth and the acceleration due to gravity.
Hydraulic jump	The abrupt change from super-critical to sub-critical flow. (A hydraulic jump is accompanied by an increase in depth.)
First flush	The initial flow of storm sewage which passes down a dual-purpose sewer following the onset of rain.
Storm tank	A tank (generally located at a sewage treatment works) provided for storage and partial treatment of excess storm sewage before discharge to a watercourse.
Sewage Analysis	
5-day Biochemical Oxygen Demand (BOD)	The amount of dissolved oxygen consumed by chemical and microbiological action when a sample is incubated for five days at 20°C. (The BOD normally gives a rough indication of the organic matter present in the sample.)
Permanganate value	Oxygen absorbed from acid, N/80, potassium permanganate, during four hours at 27°C. (The test is empirical and of limited value.)
Suspended solids	Those solids retained (after filtration of a sample) on a glass-fibre pad contained in a Gooch crucible and dried at 105°C.
Loss on ignition	The loss in weight of the suspended solids retained on the glass-fibre pad after ignition at 600°C.
Ammoniacal nitrogen	Free and combined ammonia present in a sample.

APPENDIX 2. METRICATION

In view of the forthcoming change to the metric system in the United Kingdom, we consider it proper to list the factors by means of which the British Units used in our Report may be converted to their metric equivalents. With regard both to the most appropriate units and to their abbreviations we follow the most recent official publication on the subject[1].

Conversion of numerical values

The factors by which the British Units require to be multiplied to give the metric equivalents are listed below; they are expressed to three significant figures. Conversions such as from ft/sec to m/s are not included since the factor is clearly the same as that from ft to m, but factors are included from "per in" and "per acre" as these terms occur in the Report.

	To convert	*To*	*Multiply by*
Length	in	mm	25.4
	per in	per mm	0.0394
	ft	m	0.305
	mile	km	1.61
Area	in^2	mm^2	645
	ft^2	m^2	0.0929
	yd^2	m^2	0.836
	acres	ha	0.405
	per acre	per ha	2.47
Volume	pint	l	0.568
	gal	l	4.55
	mil gal	m^3	4550
Discharge	cusec	l/s	28.3
	m.g.d.	l/s	52.6
Weight, etc.	lb	kg	0.454
	lb/ft	kg/m	1.49
	lb/mil gal	g/m^3	0.100

Conversion of Equations

Most of the equations in the Report involve British Units; a list is therefore appended of the changes in numerical factors required for conversion to metric units. This list is preceded by one giving the units used in the conversions, those in parenthesis being the British Units used in the Report. It may be noted that flow rates DWF and Q are expressed in both g.p.d. and m.g.d. in the Report. The most appropriate metric unit for dry-weather flows and overflow settings would appear to be l/s. Where (as in *Formula A*) the setting is expressed in terms of a daily volume per person, l/d seems preferable.

A	ha (acres)
a	m^2/person (yd^2/head)
D	h/mm (h/in)
DWF	l/d (g.p.d.), or l/s (m.g.d.)
E	l/d (g.p.d.)
G	l/person day (g.h.d.)
i, i_c	mm/h (in/h)
L_s, L_e	kg/a (lb/year)
Q	l/d (g.p.d.), or l/s (m.g.d.)
q	l/s (m.g.d.)
R	mm (in)
V	m^3/a (mil gal/year)
w	l/person day (g.h.d.)
Eqn. 1.	$1.84 \rightarrow 0.36$ (Q, q in l/s)
2.	$20 \rightarrow 0.79$, $8.8 \rightarrow 0.346$
3.	$0.1135 \rightarrow 2.88$, $20 \rightarrow 0.79$
4.	$0.0617 \rightarrow 8.02$, $20 \rightarrow 0.79$ (Q, q in l/s)
5.*	$20 \rightarrow 0.79$, $16.2 \rightarrow 0.125$ (q in l/s)
6.	$6 \rightarrow 0.236$
7–8.	Unchanged
9–11.	$300 \rightarrow 1360$ (Q in l/d)
12.	$300 \rightarrow 1360$ (Q, DWF in l/d)
13.	Unchanged
14.	$0.543 \rightarrow 2.78$ (Q, DWF in l/s)
15.	$0.1135 \rightarrow 2.88$, $20 \rightarrow 0.79$
16.	$0.0617 \rightarrow 8.02, 20 \rightarrow 0.79$ (Q, DWF in l/s)
17.	$6 \rightarrow 0.236$
18.	$0.017 \rightarrow 7.5$, $6 \rightarrow 0.236$
19.	$0.051 \rightarrow 0.00225$, $6 \rightarrow 0.236$
20.	$36.5 \rightarrow 0.315$ (DWF in l/s)
21.	$1/6 \rightarrow 4.2$, $0.14 \rightarrow 0.71$ (DWF in l/s)
22–23.	$0.09 \rightarrow 11.8, 0.14 \rightarrow 0.71$ (Q, DWF in l/s)

* In the expression (para. 92) following Eqn. 5, $0.078 \rightarrow 0.0144$

The three formulae of Chapter 6 thus become
Setting $(Q) = DWF + 1360\,P + 2E$ l/d *Formula A*,

$$\text{Setting } (Q) = DWF + 8.02A\left\{\left(\frac{0.79R}{T}\right)^{\frac{1}{4}} - 1\right\} \text{ l/s}$$

Formula B,

$$\text{Setting } (Q) = DWF + 11.8A\left\{\left(\frac{0.71AR}{yz\,DWF}\right)^{\frac{1}{4.4}} - 1\right\} \text{ l/s}$$

Formula C,

in each case the units of DWF being the same as of Q.

Reference

1. Ministry of Housing and Local Government. Metric Units with reference to water, sewage and related subjects. (H.M.S.O. 1968).

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